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**Committee on Water Quality**
Eighth Seminar
**US Army Corps of Engineers**
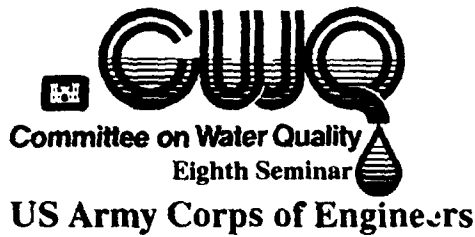
# Coastal & Inland Water Quality

Seminar Proceedings No. 22

6 - 7 February 1990
Las Vegas, Nevada

# Seminar Proceedings

# Coastal & Inland Water Quality

**6 - 7 February 1990**
**Las Vegas, Nevada**

US Army Corps of Engineers
Committee on Water Quality
Washington, DC

## FOREWORD

A two-day seminar entitled, "Coastal & Inland Water Quality," was held on 6-7 February 1990. The purpose of the seminar was to provide a forum for Corps of Engineers personnel who are routinely involved in water quality and water control work.

Topics addressed during the seminar include two papers from a plenary session, eighteen papers on Reservoir and Riverine Studies and seventeen papers on Coastal and Estuarine Studies. An appendix is also included which contains seven abstracts by individuals displaying materials during the poster session.

The seminar was co-sponsored by the Hydrologic Engineering Center of the Corps Water Resources Support Center and the Committee on Water Quality. The seminar proceedings and the general coordination of the seminar were organized by Mr. R.G. Willey of the Hydrologic Engineering Center (HEC). Expert word processing skills were provided by Ms. Kimberly Watkins-Robinson and Chris Brunner also of HEC. Review and grammatical editing were provided by contract with Ms. Debra Milligan.

Valuable assistance for coordination of the separate sessions was graciously provided by Mr. Dave Buelow and Ms. Lynn Lamar, HQUSACE; Dr. Robert Engler, WES; Mr. Jan Miller, NCD; Mr. Warren Mellema, MRD; Dr. Bolyvong Tanovan, NPD; and Mr. Dennis Barnett, SAD. The conference rooms, individual rooms and all local arrangements were organized by Mr. Robert Stuart, Los Angeles District and Mr. Mort Markowitz, SPD.

The views and conclusions expressed in these proceedings are those of the authors and are not intended to modify or replace official guidance or directives such as engineering regulations, manuals, circulars, or technical letters issued by the HQUSACE.

R. G. Willey
Editor

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**PLENARY SESSION**

# A GLANCE BACK -- A LOOK AHEAD

by

Mark Anthony[1]

In looking back, I can only conclude that I have been a very lucky guy. Early on, I became captivated by the mystic of biology and formal training compelled me to pursue aquatic biology as a career. I later learned to enjoy applied science and a teamwork approach for solving problems. Then, by happenstance, I became obsessed by the relationship of water control and water quality and by the blatant sins of the Corps of Engineers. Finally, I was recruited by the Corps to help solve problems and to be the devil's advocate. And, I have gained full appreciation for Teddy Roosevelt's statement, "Far and away the best prize that life offers is the chance to work hard at work worth doing." I can assure you that my plate has been full. I've had lots of playing time, the games have been exciting with more wins than losses, and the entertainment provided by the bureaucracy has been a bonus. Now, this opportunity to address a group that I have worked with for so many years, and for which I have such profound respect, is another bit of luck. I'm going to brag a little, compare past with present, attempt to make a point, and say goodbye.

In 1967 I was recruited by J. T. Mitchell, Jr., to plan, develop, and implement a water quality control program as a part of water control in the Ohio River Division (ORD). I just happened to be at the right place and time with barely adequate field experience and past mistakes -- pure luck! I was excited. I'd found a unique opportunity to apply a concept, a multi-disciplinary approach that included limnology, chemistry, biology, hydraulic and hydrologic engineering to solve a magnificent series of water quality and water management problems.

I quickly documented many pages of objectives, detailed approaches, and the necessary resources. I just as quickly learned about the real challenge: the bureaucracy. Being a biologist with uppity ideas and the vanguard of what the Engineering Division considered, at best, an irresponsible movement, this was not an ego trip. Then there was Al Cochran, the Chief of H&H at Office of the Chief of Engineers (OCE), who insisted on a format called the Technical Studies Work Plan for proposals such as mine. Nevertheless, I was allowed to recruit Glenn Drummond, Don Robey, and Gary McKee. We rode the crest of environmental legislation, and a wave of Corps criticism, and we helped change the way things were being done at the Corps.

Believe me, those were heady times. We wrote our own guidance for pre- and post- impoundment studies, encouraged and bluffed Corps districts to staff water quality expertise, and started a water quality lab in ORD. We also struggled with the Waterways Experiment Station (WES), OCE, the districts, and with our own design element, but, at the same time we gained support and made friends. Within a few years, Robey and Drummond had performed some 35 model applications, including 26 selective withdrawal structure designs. The districts were well up the learning curve and we had acquired excellent credibility with State and Federal agencies.

Flextime in those days was getting eight hours at home. For instance, our first model applications were run on a computer in Baltimore on weekends. Then we used Wright Patterson Air Force Base after midnight. Later, Robey and Drummond only had to walk down the street a few blocks to use a computer. They often got home by 9:00 or 10:00 p.m.

I will try to summarize the results of products of 23 years of effort in a few words. In spite of widespread, chronic sources of pollution, the discharges from most of our 78 storage projects meet stream standards most of the time. Most projects support quality fisheries. The operation of individual projects and reservoir systems has led to significant recovery along hundreds of miles of downstream reaches. We have

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[1]Private Consultant (Formerly Chief, Water Quality Branch, U.S. Army Corps of Engineers, Ohio River Division)

avoided litigation and enjoy a high level of credibility with our customers. I was also lucky to experience the challenge of the 1988 drought. Our water control capability was thoroughly tested, and our water quality control approach proved to be valid and adequate.

We have been successful because we are organized so as to employ the necessary teamwork to identify the problems, design the studies, collect and interpret the data, apply engineering judgments, perform risk analysis, implement the solutions, and evaluate the success of each solution. It sounds a lot like life cycle management, doesn't it? However, I again want to emphasize that everything that has been accomplished in ORD is strictly the result of teamwork. The district players are the stars on this team. We have continuously benefited from the people and the products at WES and the Hydrologic Engineering Center, as well as the Cold Regions Research and Engineering Laboratory: physical modeling, peer review, more detailed concepts, new techniques, better procedures, math modeling, workshops, and training.

I must also cite the players at Headquarters. Sam Powell would never admit the role he played before Elker arrived on the scene. Then there are the products of the Water Quality Committee, including this seminar series. I include many of you. By supporting Corps research programs, and by participating and sharing experience, problems, and mistakes, you have contributed to the Corps' body of knowledge. You have been a source of ideas. Then, I must mention the Edinger's, Buchak's, and Orlob's and all the scientists and engineers from State and Federal agencies and universities who have worked with us over the years in solving problems.

Now I'll focus my spotlight and make my point. Collectively and individually, you face far greater challenges, more rigorous demands, more complex problems, and a much greater variety of issues and frustration than we did 23 years ago. Our problems were relatively simple; our objectives relatively straight-forward; the parameters were more easily measured; and our solutions were less controversial and more feasible. You require a far greater number of specialized disciplines in order to select and apply precise techniques and the penalties for mistakes are quite painful. Air pollution, landfills, HTW, water quality, wetlands, water resources -- you must cope with an unbelievable number of special interest g.oups who have conflicting needs and agendas. And finally, in spite of our frustrations, the Corps was fun, motivating, and highly professional in 1967. You must cope with an increasingly confused, demoralized, and disorganized Federal bureaucracy.

My suggestions to you about how to cope with the challenges you face may appear mundane and trivial or, perhaps, too obvious to bear repeating. Nevertheless, they work. I must point out that all of us become careless or sloppy at times, or allow pressure and frustration to become an excuse. "Going back to basics" is an expression used by John Edinger to describe his habit of reviewing "Limnology" by Welch when he is struggling with a technical problem or a mental block. I strongly recommend this approach as therapy or a way to step back and refocus. I'm sure that all of you have a similar crutch or two on your bookshelf.

Secondly, have you carefully defined and spelled out objectives and priorities? Are you asking the right questions? Are you ready to drop low priorities, mesh efforts, or phase activities in or out? Have you exposed your plans to adequate peer review?

I think that the issue of essential disciplinary specialties and adequate teams is especially critical. Are you taking full advantage of the expertise available in other elements? The most significant product of millions of dollars of research is the superb level of expertise in our labs. Are you using the phone, the fax, and the bulletin boards? These people can provide skilled peer review, instant help, and advice about how to do better with less. With cuts looming, you can't afford duplication of effort or wheel-spinning and you must close ranks. If you have a choice between outside help and our laboratories, I urge the latter. If you really have your act together, it will be more effective and cheaper over time. If you are not certain about who knows what, now is the time to get acquainted with both lab folks and other divisions and districts who share similar problems. This group reflects a massive body of knowledge and experience. Many of us know alot about how the informal organization works.

The urgency that you face for research, basic and applied, is yet another reason to close ranks.

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You simply must become more sophisticated and proficient in helping identify Corps-wide research needs and priorities. Budget cuts, FTE reductions, and growing needs mandate more effective participation on the part of field office personnel, Field Review Group Reps, R&D coordinators, laboratory personnel, and Tech Monitors in this process. In spite of the constraints, most of the improvement must come from the field office and Field Review Group level. How? Quit being parochial.

I know for a fact that most of you are where you are because you love what you do. Most of you are skilled, dedicated, journeymen scientists. Sometimes your dedication and pride lead you to assume that your problems are unique or that only you can solve them. You must take the time and effort to look over the significant level of help, procedures, and technology that is available. If you are a skilled problem-solver, you are already using available technology. There is a significant degree of Corps-wide commonality regarding research needs. You cannot afford to support research work units that fail to meet the criteria. A valid program demands much more than a simple yes or no vote each year.

# UTILITY OF SEDIMENT QUALITY CRITERIA (SQC) FOR THE ENVIRONMENTAL ASSESSMENT AND MANAGEMENT OF DREDGING AND DISPOSAL OF CONTAMINATED SEDIMENTS

by

James M. Brannon, Victor A. McFarland,
Thomas D. Wright, and Robert M. Engler[1]

## INTRODUCTION

The Corps of Engineers (CE) has the mission to maintain, improve, and extend the navigable waterways of the United States. Furthermore, the CE has the lead in regulating the disposal of dredged material into inland and ocean waters and conducting appropriate ecological assessments prior to selecting disposal alternatives. To carry out this mission, the CE is responsible for the dredging and disposal of large volumes of sediment each year. The CE annually dredges about 230 million cubic yards (c.y.) in maintenance and about 70 million c.y. in new dredging operations. In addition, 100-150 million c y of sediments dredged by others each year are subject to permits issued by the Corps (Engler et al. 1988). The CE regulatory responsibilities involve review of some 10,000-30,000 permit applications each year as well as appropriate maintenance of, and improvements to, the 25,000-mile congressionally authorized Federal navigation system serving 42 of the 50 states (Engler et al. 1988).

The CE's authority stems from Section 10 of the River and Harbor Act of 1899, Section 404 of the Clean Water Act (Public Law 92-500, as amended), and Section 103 of the Marine Protection, Research, and Sanctuaries Act, ("Ocean Dumping Act", Public Law 92-532, as amended). The guidelines for disposal of dredged material pursuant to Section 404(b)(1) of the Clean Water Act and Section 103 of the Ocean Dumping Act require compliance with several conditions prior to allowing disposal of dredged material in inland or ocean waters. Compliance requires the avoidance of "unacceptable adverse effects" to the aquatic environment. The 404(b)(1) guidelines at 40 CFR, Part 230, and ocean dumping criteria at 40 CFR, Part 220-228 provide general regulatory guidance and specific objectives and testing protocols for evaluating sediment that must be dredged for navigation purposes. Management guidance is provided in 33 CFR 209, 335-338, and additional CE policy guidance is found in 33 CFR 320-330.

The U. S. Environmental Protection Agency is currently authorized to develop and implement SQC under Section 304(a) of the Clean Water Act. SQC may be applied in many regulatory decisions, including identification of problem areas, source control, establishment of cleanup goals, development of discharge and dumping permit criteria, and determination of monitoring requirements. Therefore, SQC, when promulgated and if used for dredging and disposal, will profoundly affect Corps dredging and disposal operations as it is likely that aquatic disposal of dredged material and selection of disposal alternatives will be based on SQC.

Currently both Federal and State regulatory agencies are developing SQC in order to identify contaminant levels in sediment that are causing adverse effects in the environment. The notion is, however, not new, as SQC were promulgated in the late 1960's and early 1970's (Engler 1980) and were found to be technically inadequate for use in assessing and managing dredged material disposal. The technical inadequacies were related to the profoundly complex geochemical nature of dredged material and the SQC's inability to address broad ecological consideration set forth in pertinent environmental legislation, e.g., the Clean Water Act and the Ocean Dumping Act. Recently, however, the approaches

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used to evaluate sediment quality have focused on determining the relationships between contaminant concentrations in sediment and adverse biological effects resulting in derivation of numerical SQC for specific contaminants.

The CE is concerned with the direction of development in SQC because of the possible impacts that implementation of such criteria would have on the currently used effects-based approach for assessing dredging and disposal of dredged material. Furthermore, the effects based approach has been selected by the CE and the EPA (Engler 1980) as it clearly addresses the ecological considerations set forth in the environmental legislation.

This paper presents an overview of several SQC methods under development, a discussion of how the CE presently evaluates dredged material, and the possible impacts of sediment criteria on dredging and disposal. Emphasis is placed on the utility and applicability of the various approaches to assessment of contaminated sediments and dredging and disposal.

## SQC DEVELOPMENTAL APPROACHES

Numerous SQC approaches are presently under development or being considered or evaluated for use. These include background sediment chemistry, spiked sediment bioassay, interstitial water toxicity, tissue residue analysis, sediment/water equilibrium partitioning, sediment bioassay, screening level concentration, apparent effects threshold, and the sediment quality triad. Some approaches are descriptive, i.e., not chemical specific and cannot be used alone to generate numerical SQC, while others are purported to be chemical specific (numeric). Additional methods consist of mixtures of descriptive and numeric approaches. A brief description of each major approach currently under development is provided with a discussion of technical utility and its advantages and disadvantages. More detailed summaries of the various SQC approaches may be found elsewhere (Chapman 1989; Shea 1988; Neff et al. 1988; Battelle 1988; Tetra Tech 1985; Long and Chapman 1985).

### Background Chemical Concentration

This method consists of comparing chemical concentration values in sediments to be dredged and disposed with chemical concentrations from sediments in the same geographic region that are thought to represent background conditions. This approach does not address ecological cause and effect considerations, but establishes the criteria in relation to contaminant levels in a reference area. The methodology used to derive background levels of contaminants can vary from analysis of surficial sediment grab samples to deep cores intended to indicate background contaminant levels of chemicals that existed prior to industrial activities.

An example of this type approach was the establishment of decision criteria for the suitability of dredged material disposal at the Four Mile Rock Disposal site in Puget Sound, Washington (U. S. EPA/WDOE 1984). The EPA and State agencies adopted the position that no further chemical or biological degradation of the Four Mile Rock disposal site would be allowed. If materials intended for disposal were more chemically contaminated than sediments currently at the site, open-water disposal at this site was not permitted.

The Background Chemical Contamination approach is frequently advocated because it is easy to implement, provides numerical limits, and does not require an understanding of any of the relationships between sediment geochemistry and biological effects. Conversely, the limitations and disadvantages of this method are many. Chapman (1989), for example, mentioned: (1) difficulty of selecting reference sediments, (2) site-specific nature of the criteria generated (dependent largely on stations chosen to represent background), and (3) inability of the method to distinguish bioavailable chemicals from those that are not bioavailable. JRB Associates (1984) listed an additional disadvantage, the fact that the criteria generated may be too restrictive since there is no attempt to define a maximum, biologically safe level of contamination.

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The definition of "geographic region" also poses problems with this method. There are difficulties involved with deciding the limits of applicability for such criteria. For example, the "Jensen Criteria," which rated natural background metal concentrations in pristine areas as "highly polluted" were applied over the entire Great Lakes region (Engler 1980). Further, high levels of organic material of natural origin in uninhabited areas also lead to the classification of "highly polluted" (Wright 1974). In addition, this method does not satisfy the requirement under Section 103 of the Ocean Dumping Act that criteria for disposal of dredged material must be effects-based. This results in significant difficulty in technically defending criteria derived in such a manner (Chapman 1989). However, comparison of total sediment concentrations at the dredging site and the disposal site is a frequently useful inventory of potential contaminants of concern and is used by the Corps as part of its "reason to believe" evaluation, which consists of reviewing available material to determine if contaminants are present (Engler 1988). Chemical inventories can identify contaminants of concern and trigger effects-based evaluation and will be discussed later in this paper.

## Spiked-Sediment Bioassay

This approach attempts to ascertain cause-and-effect relationships between an individual chemical and a biological response by using spiked sediment and bioassay procedures. The results would be used to develop numerical criteria quantifying the case and effect relationship. The main difference between this approach and the sediment bioassay, using naturally contaminated sediment, is that test organisms are exposed to sediments that have been spiked in the laboratory with known quantities of potentially toxic chemical s or mixtures of chemicals. At the end of a specified time period, the response of the test organism is examined in relation to a biological endpoint and results are analyzed in the manner described for sediment bioassays.

Inherent in the spiked-sediment bioassay approach is the assumption that spiked sediments and contaminated sediments in the environment will act similarly. This may not always be the case, especially for metals that exist in a wide range of sediment phases of varying biological availability (Brannon et al. 1976; Engler 1980). For example, the majority of some metals in sediment may be associated or co-precipitated with iron and manganese oxides or incorporated in the structure of minerals. Spiked metals could not mimic the behavior of metals present in sediment in such chemically and biologically unavailable forms.

Technical problems in relating spiked sediment responses to that of an *in situ* sediment with similar contamination can also result from factors other than the phase association of the contaminants. Behavior of both metals and organic contaminants added to sediment s can be affected by the method of spiking, equilibration time for the spiked compound, and sorption to container walls (Word et al. 1987). No relationship has been developed between biological responses for spiked sediments and *in situ* or "real world" contaminated sediments. Changes in toxicity and bioavailability may occur in "real world" sediments that cannot be duplicated in the laboratory because of the complex array of environmental conditions that exist at any contaminated sediment location. This method is therefore unsuitable for dredged material disposal evaluations. It is, however, a useful research tool for examining the effects of sediment contaminants and may lead to a better understanding of sediment/contaminant/biota relationships.

## Equilibrium Partitioning Approach

The equilibrium partitioning (EP) approach involves estimating the concentration of a contaminant in interstitial water from contaminant concentrations in the sediment, then comparing the calculated interstitial water concentrations to water quality criteria. If the predicted sediment interstitial water concentration for a given contaminant exceeds its respective chronic water quality criterion, the sediment would be expected to cause adverse effects. Conversely, the water quality criterion could be used as starting point to back-calculate and determine the related sediment concentration. Presently, the EP approach is applicable only to no-polar hydrophobic compounds because partitioning of such compounds between sediment and water has been shown to be highly correlated with the organic carbon content of sediments. Partitioning of metals from sediment into interstitial water is not sufficiently

well understood to permit calculation of interstitial water metal concentrations. Approaches based on modeling the interactions between heavy metals, iron and manganese oxides and organic carbon are under development. A new approach based on acid-volatile sulfides is also under development. However, none of these variations are capable of making predictions of interstitial water metal concentrations. Mechanisms that control the partitioning of polar organic compounds are also poorly understood.

The EP calculation procedure for non-polar hydrophobic organic compounds is based on the relationship:

$$rSQC = K_p * cWQC$$

where

cWQC   =   Water Quality Chronic Criterion (ug/L)
rSQC   =   Sediment Quality Criterion (ug/kg sediment)
$K_p$       =   Partition coefficient for the chemical (L/kg) between sediment and water

For particles with fraction organic carbon ($f_{oc}$) > 0.005 organic carbon by weight, organic carbon appears to be the predominant sorption phase for non-polar hydrophobic organic compounds. Therefore, if $K_{oc} \approx K_{ow}$, then $K_p = f_{oc}K_{ow}$, where $K_{ow}$ is the octanol-water partition coefficient and $K_{oc}$ is the particle organic carbon partition coefficient (L/kg organic carbon).

Underlying assumptions of the equilibrium partitioning approach (Chapman 1989) are that a chemical establishes equilibrium between the interstitial water and sediment-associated fractions, that the distribution can be described by a partition coefficient between any two phases, sediment contaminant concentration, and total organic carbon. This also assumes that the only toxic fraction of a sediment contaminant is that fraction freely dissolved within interstitial waters (Word et al. 1987). Furthermore, this method assumes that benthic organisms will display similar responses to contaminants in the interstitial water as did the water column organisms used to derive water quality criteria.

The main advantage of the EP approach is that existing water quality criteria can be used, allowing use of the large existing data base relating contaminant concentrations in water to effects on aquatic organisms. If prediction of interstitial water concentrations based on sediment properties and contaminant concentrations for a wide range of contaminants becomes possible, then the approach is applicable for most types of sediments and aquatic environments. This approach could rapidly evaluate sediments, since rSQC could be calculated as soon as sediment chemistry data were available. For a chemical of concern, the EP permits an assessment of whether contaminant concentrations are approaching an effects level.

The major disadvantage of the EP approach is that criteria based on biological effects cannot be directly developed for contaminants for which water quality criteria are not available. This is reflected in the fact that interim sediment criteria presently exist for only 12 contaminants. In addition to its limited applicability (nonpolar organic compounds for which water quality criteria exist), the approach also does not use toxicological data developed from the sediment of interest and cannot address possible interactive and antagonistic effects of contaminants. Krueger, et al. (1989), has also claimed that a large degree of uncertainty exists for EP values calculated for individual contaminants.

The CE is charged with dredging and disposal of sediments containing a wide range of contaminants. Once SQC are developed, a sediment below all existing SQC limits could nevertheless have contaminants that would give "reason to believe" that the sediment could adversely impact the environment. Regulators faced with such a situation would need bioassays to determine the suitability of the material for disposal. Even if SQC based on EP are developed for a wide range of chemicals, it remains a chemical-by-chemical approach that cannot evaluate interactive or antagonistic effects. Therefore, it is highly likely that bioassays would still need to be conducted to resolve equivocal EP data.

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If the EP does result in the derivation of SQC values with a large degree of uncertainty, other questions arise. If a sediment passes SQC for most chemicals, but is above limits for others, how will the sediment be evaluated? Will it be determined to be unsuitable for various disposal alternatives or will bioass ys be conducted to determine the integrated environmental impact of the sediment on aquatic organisms? If bioassays are conducted and the sediment shows no potential adverse impacts, will the disposal of the material be regulated by the bioassay results or the SQC criteria? Such issues, perhaps more procedural than technical, point out some of the problems raised by chemical-by-chemical approaches such as the EP that do not consider interactive and antagonistic effects of contaminants. SQC that are overly restrictive of aquatic disposal because they do not accurately predict biological effects may require selection of other alternatives, e.g. land-based, that are not addressed by these criteria and may actually be less environmentally preferable alternatives.

Interstitial Water Toxicity

This approach consists of three phases: (1) identification of the physical and chemical nature of constituents potentially causing acute toxicity, (2) fractionation schemes and analytical methods to identify the toxicants, and (3) procedures to confirm that the suspected toxicants are the cause of toxicity.

The first step in this procedure consists of isolating interstitial water (IW) from sediment samples followed by toxicity tests on the isolated pore water. A Toxicity Identification Evaluation (TIE) (Burkhard and Ankley 1989) is then conducted in an attempt to identify the chemicals responsible for acute toxicity. The greatest advantage of this approach is that it may allow the potential identification of the IW components responsible for toxicity. However, only limited work, mainly on industrial effluents, has been conducted using this recently developed approach.

This approach possesses a significant number of potential disadvantages and needs substantial development before consideration as a regulatory tool. Specifically, there are no standard methods for separating IW or maintaining its geochemical integrity. In addition, the relationship between sediment IW toxicity in situ and the toxicity of extracted sediment pore water needs to be explored for different classes of compounds. The complex and operationally-defined methods used to isolate the interstitial water and perform the toxicity tests on the isolated pore water may lead to artifacts that could result in numerous erroneous conclusions about the toxicants. The tests used to identify toxic chemicals in the IW are cumbersome and cannot comply with the effects-based approach mandated in the applicable environmental regulations. Generally, no single test can be used to confirm a suspected chemical as the only or even primary toxicant; it is therefore necessary to use multiple confirmation procedures. This difficulty can be exacerbated by differing substances causing toxicity in multiple sediment samples from a supposedly homogenous source.

IW toxicity may have potential as a research tool for identifying toxic components in IW and for developing a better understanding of the complex biogeochemical relationships between IW and the whole sediment matrix. The cumbersome chemical fractionation procedures involved and the present low state of development make IW toxicity unsuitable for the regulatory process. In order to be successfully implemented for regulation, a method must be a simple, cost-effective, technical assessment of sediments proposed for dredging and disposal.

Sediment Quality Triad

The triad approach (Chapman 1986) consists of three separate but intrinsically related components, (1) sediment contaminant concentration measurements, (2) sediment toxicity bioassays , and (3) benthic community structure assessment. This approach, using a combination of some of the previously discussed procedures, seeks to independently measure sediment contamination, sediment toxicity, and biological alteration, and then use the burden of evidence to assess sediment quality based on all three sets of measurements.

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The triad approach combines all bioassay and *in situ* biological effects data to derive a single value. *In situ* bioeffects measures used to date include benthic infaunal community structure and bottom-fish histopathological abnormalities (Chapman 1989). Biological effects must be defined as either minimal or severe to establish criteria in terms of chemical concentrations below which biological effects would be expected to be minimal and above which biological effects would be expected to be severe. Standards in the form of sediment quality criteria can be set from the contaminant concentrations that are always associated with effects.

The major perceived advantage of the triad approach (Chapman 1989) is that the combination of the three separate measures in the triad in a burden-of-evidence approach allows for differentiation of toxicity related to contamination from natural variability and/or laboratory artifacts. Other advantages are that the approach does not require a *priori* assumptions concerning the specific mechanisms of interaction between organisms and toxic contaminants.

This method rests on two assumptions: (1) use of large data bases adequately allows for interactions between contaminants in complex sediment mixtures, actions of unknown toxic contaminants, and for all other environmental factors that influence biological response in addition to toxicant concentrations; (2) the triad also assumes that different endpoints of different bioassays, selected chemical contaminants, and selected measures of benthic community structure are appropriate indicators of bioeffects and can be treated in an additive manner, with each having equal weight (Chapman 1989). The method may be useful in a relative comparison of one sediment location to another with a potential ranking of locations.

Perceived disadvantages of the triad (Chapman 1989) for numerical SQC development include lack of statistical criteria for developing numerical criteria, no rigorous criteria for calculating the single SQC value from each of the three triad components, need for a large and expensive database, possible influence of unmeasured toxicants that may or may not covary with measured chemicals, and choice of a reference site that is often made without adequate information regarding how degraded that site may be. Limitations of the triad approach for dredging and disposal operations are discussed below under "Limitations of the Triad and AET."

Apparent Effects Threshold (AET)

As with the triad, the AET is a three compartment approach that uses sediment chemical analysis for a suite of chemicals, benthic faunal analysis, and bioassay methods. The benthic faunal analysis and bioassays are used to determine sediment toxicity or impact in order to detect similarities or dissimilarities between or among discrete sediment sample locations. Following the initial characterization, the next, and most technically dubious step consists of correlating individual chemical concentrations with effects demonstrated by the two biological assessments. Subsequent use of the derived AET chemical concentration would then purport to signal a known or documented effect. The AET for a given chemical is the sediment concentration above which a particular adverse biological effect is always statistically significant ($p \leq 0.05$) relative to appropriate reference conditions. The AET approach is similar to the triad in that it attempts to relate concentrations of contaminants in sediments to toxicity bioassays and benthic community structure.

In most cases the AET has been used to develop two sets of sediment quality values, one of which identifies low chemical concentrations below which biological effects are improbable, and a second set of values that identifies higher chemical concentrations above which multiple biological effects are expected. Direct biological testing of sediment is needed between the two extremes.

The AET approach rests on three main assumptions. The first is the same as assumption (1) for the triad, the second is that appropriate indicators of bioavailability and toxicity can be determined based on field and/or laboratory data, and the third is that correlations can be used to relate sediment concentrations to biological effects without knowledge of the interactions that occur between sediment contaminants, toxicity, and faunal abundance. Disadvantages of the AET include those listed for the triad. In addition, an AET may identify both problem and nonproblem sediments as being of concern.

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## Limitations of the Triad and AET

Benthic community structure is a component of both the triad and AET. Community structure determinations are a "snapshot in time" and the species present are influenced by the events, and particularly the extreme events, that have occurred over some period of time. In long-lived animals, that period may encompass several years. Animals living at a site integrate all of the environmental perturbations that occur and such integration can be used to assess the overall condition of a particular environment. However, navigational dredging projects are most commonly found in highly industrialized/urban areas which have a wide variety of physical and chemical perturbations and where navigation itself is a significant form of perturbation that has a profound influence on benthic community structure.

The physical effects of traffic and of repeated dredging in a channel may eliminate or severely reduce the benthic fauna, particularly in navigation channels. These perturbations have nothing to do with sediment-associated contaminants. Likewise, water quality degradation from outfalls, thermal discharges, surface runoff, and a host of other perturbations may confound any effects of sediment contaminants. Therefore, if an "unhealthy" fauna (comparison to reference) is found, bioassays will be necessary to evaluate the sediment for disposal without confusion from physical effects, time, water column effects, or episodic events. The question then arises, "Why not use bioassays in the first place?"

Reference areas chosen for relating AET's and the triad to test sediments do not represent the physical, chemical, and biological conditions of the test sediment locations. Even though the reference area may be uncontaminated and free from human activities, it may not reflect important noncontaminant influences on biological diversity and abundance that are operant at the test site. Comparisons of test sediment data to such reference areas will provide little information of technical value because contaminant effects cannot be separated from natural variability.

Both the triad and AET were developed, in part, using toxicity data from 48-hour, bi-valve larvae and amphipod sediment bioassays. There is evidence that both of these bioassays may be highly sensitive to grain-size and the total organic carbon content of the sediment (Long and Buchman 1989). Some of the test organisms used in the triad and AET bioassays are from sandy habitats and are not well adapted to surviving in mud or mud suspensions. Therefore, both the triad and AET draw conclusions about toxicity that may have other ecological interpretations (Spies, 1989). This leads to the high probability of false positives on sediment toxicity.

The EPA Science Advisory Board recommended to the EPA Administrator in 1989 that the AET approach should not be used to develop general, broadly applicable sediment quality criteria. Reasons for this recommendation were the site specific nature of the approach, its inability to determine cause and effect relationships, its lack of independent validation, and its inability to describe differences in bioavailability of chemicals in different sediments. The triad approach suffers from many of these shortcomings.

## Tissue Residue Approach

In this approach, sediment chemical concentrations are determined that will result in residue levels in exposed organisms that would not be harmful to the organism itself or to its predators. The linkage of sediment contaminant concentrations to residues in organisms is made through measurements at a test site, equilibrium partitioning-based predictions, or steady-state food chain models.

Many difficulties exist with this approach because of the varied metabolic strategies and needs of aquatic organisms. For example, high concentrations of heavy metals are normal in many aquatic invertebrates, such as copper in crustaceans or zinc in oysters. In the case of organic compounds,

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metabolites may be either more or less toxic than the parent compounds, or the presence of more abundant but less toxic compounds of similar structure may reduce toxicity by competing for receptor sites.

Because interpretation of tissue residue data are largely subjective in situations other than when FDA limits exist, this method is of limited applicability to dredging and dredged material disposal. Tissue residue comparisons can be useful, however, when coupled with the background approach. This involves comparing predicted or empirical tissue residues from dredged material to background tissue residue concentrations at the disposal site to determine if dredged material disposal would result in increased contaminant tissue concentrations at the disposal site. An example of this approach is the New York District Matrix.

## Screening Level Concentration (SLC)

The SLC approach (Neff et al. 1988) is a field-based method that estimates the highest concentration of a particular contaminant in sediment that can be tolerated by approximately 95% of benthic infauna. In the SLC, the presence of selected benthic indicator species is matched with the total bulk sediment concentration of particular individual contaminants to determine the concentration of each contaminant at which 90% of the particular individual species were present. This concentration is called the species screening level concentration (SSLC) and SSLCs are calculated for all appropriate species for each contaminant. SSLCs for all species are compared with sediment concentrations of each measured contaminant to determine the contaminant concentration above which 95% of the SSLCs occur. This final concentration is called the SLC. Data required for computation of SLC values are 20 stations for each SSLC; 20 taxa for each SLC, a gradient of contamination, and a similar level of taxonomy at each station.

The SLC relies on the assumptions: (1) if the database is large then factors other than contamination that influence benthic infaunal distributions do not need to be considered; (2) the presence of a species at a site implies the lack of a biological effect due to contamination; and (3) no assumptions are required concerning specific mechanisms of interaction between organisms and contaminants.

The SLCs greatest disadvantage is that results can be strongly influenced by the presence of unmeasured toxic contaminants that may or may not covary with measured chemicals, interactive effects of chemicals, and by other sediment characteristics. Other disadvantages include the wide range of field data needed, the great effect of the range and distribution of contaminant concentrations and particular species on SLCs, lack of selection criteria for species, and lack of a mechanism for separating single contaminant effects from combined contaminant effects. Many of the problems incurred with the community structure determination, discussed in relation to the triad and AET, also affect the SLC approach.

## Sediment Bioassay Approach

The sediment bioassay approach is an effects-based method and is the method currently required by USEPA regulations for the CE to evaluate possible environmental impacts at aquatic disposal sites (EPA/CE 1977; CE 1976). In this approach, test species are exposed in the laboratory to sediments proposed for disposal and the response of the organisms is evaluated in relation to reference organism responses in terms of a specified biological endpoint such as mortality, bioaccumulation, growth, reproduction, etc. The precise chemical composition of sediments need not be known to use this approach as it is a "whole sediment" test. Chemicals that may go undetected analytically are covered in the bioassay approach. Potential biological effects are, therefore, appropriately estimated because dredged sediments are managed on a "whole sediment" basis.

The advantage and logic of this approach is that the effects of sediment contaminant interactions such as synergism, additivity, and antagonism are integrated by the response of the organisms. In addition, this approach is consistent with the regulatory requirements and methods used to develop

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water quality criteria (effects-based bioassays), and thus render the approach and rationale technically acceptable and defensible (Chapman 1989).

Chapman (1989) has listed a number of disadvantages of sediment bioassays for the development of SQC: (1) because sediments may contain mixtures of covarying toxic contaminants, bioassays do not provide chemical-specific sediment quality values; (2) there is a lack of standardized methods for sediment bioassays, dosing sediments, and measuring interstitial water concentrations; and (3) difficulty to implement the approach because procedures are not well-developed for many species of organisms and the available tests may not reflect chronic effects. However, sediment acute toxicity bioassay test procedures have been developed and are well documented for testing sediments, although most methods need better field validation. Field validation was recently conducted in the CE/EPA Field Verification Program (Peddicord 1988) and demonstrated the utility of "whole sediment" toxicity tests.

The advantages of the sediment bioassay approach include ease of use, relative cost effectiveness, flexibility, widespread acceptance, and compliance with regulatory requirements. Sediment bioassays were adopted for use under EPA regulations specifically because this approach addresses considerations set forth in the cited regulations for dredged material (Engler 1988). Acute toxicity sediment bioassay testing is an effects based approach that takes into account the fact that sediment is a complex mixture of substances whose potentially interactive effects cannot be predicted through chemical analyses. Bioassay methods can also be used to evaluate the effects that sediments exert *in situ* as well as to identify suspected sources of sediment pollution.

## THE CURRENT REGULATORY APPROACH

Three specific situations or reasons exist for evaluating sediments. The first is the necessity for disposal of navigation channel dredged sediments. For navigation channel sediments the only question to be answered is what unacceptable adverse effects, if any, will dredged material have in a particular disposal environment. EPA regulations at 40 CFR, Parts 220-228, and 40 CFR, Part 230, provide regulatory guidance on aquatic disposal of dredged material. The second reason to evaluate sediments is to determine what effects on aquatic ecosystems they may have if they are left undisturbed or if they are removed for environmental purposes. If sediments are determined to be exerting unacceptable environmental effects, consideration may then be given to some type of remediation, which may or may not include removal. If sediments are to be removed, the potential effects that these sediments will exert at the disposal site must be considered. The third reason, recently advanced by the EPA, is for source control of contaminants. Determination of places where sediments, as sinks for contaminants introduced into the aquatic environment, are exerting unacceptable environmental impacts could lead to identification of the source of the contamination.

Most criticism of the present regulatory approach to evaluating sediment for dredging and disposal centers on the acute toxicity bioassay endpoint, which is usually mortality. However, active development of existing bioassays to include sublethal effects is ongoing. The method has also been criticized because it does not identify the causative agents of observed effects and therefore cannot be used in establishing limits for contaminants in discharged materials.

For dredged material disposal, the greatest need is for methods of disposal alternative evaluation. Other than for unconfined open water disposal, disposal alternative evaluations will not be assisted by promulgation of technically valid SQC. The CE has developed a dredged material management (Testing Protocol) strategy (Francingues et al. 1985), the objective of which is to address testing and disposal of contaminated dredged material. This strategy has been adopted as CE policy and is incorporated by reference in 33 CFR, Parts 209, 335-338, 26 April 1988 (Corps Dredging Regulation). The steps set forth in the strategy for managing dredged material proposed for any disposal alternative are as follows;

(1) Evaluate contamination potential
(2) Consider potential disposal alternatives
(3) Identify potential problems

(4) Apply appropriate testing protocols
(5) Assess the need for disposal restrictions
(6) Identify available options to control adverse environmental impacts
(7) Evaluate design considerations
(8) Select appropriate control measures to prevent adverse environmental impacts

A tiered testing approach for assessing materials proposed for aquatic disposal is described briefly below, and is detailed in Engler, et al. (1988). An example of the application of the current approach, taken from Engler, et al. (1988) is given in Table 1.

The initial (Tier 1) evaluation consists of reviewing available information to establish whether there is a "reason to believe" that contaminants are or are not present. If there is reason to believe that contaminants are present, or if insufficient information exists to allow adequate evaluation, a second tier follows. The second tier of testing addresses benthic concerns and consists of a bulk sediment analysis in order to compare the chemical composition of the dredged material to the composition of the material at the disposal site. If substantially greater concentrations are observed in the dredged material and there is reason to believe that the substances are bioavailable, a third tier of testing may be required in which benthic impacts are addressed.

If there is concern regarding impacts to benthic organisms, an acute toxicity benthic bioassay testing three species may be conducted. If there is reason to believe that bioaccumulation is of concern, a second component of the third tier consists of evaluating the potential uptake of contaminants in either the field or laboratory. Strengths and weaknesses of the present regulatory approach, which relies on bioassays, are the same as those presented earlier in the discussion of sediment bioassays.

For water column impacts, an elutriate test may be performed to evaluate contaminant release into dredging or disposal site water and for comparison with water quality standards after consideration of mixing. A suspended phase and/or liquid phase bioassay may also be conducted if there are no water quality criteria or the standards are thought to be inadequate or inappropriate.

## CONCLUSIONS

This review has shown that SQC approaches under development possess significant shortcomings in relation to ecological assessment of dredging and disposal operations. If SQC approaches can incorporate cause-and-effect relationships and be improved to the point where results are a reliable predictor of potential impacts at the disposal site, then SQC will be useful for evaluating the potential effects of aquatic disposal or perhaps other alternatives, as appropriate. The current state-of-the-art is not sufficient for the promulgation of SQC for the ecological assessment and subsequent regulation of dredged sediments. The EPA's Science Advisory Board subcommittee on sediment criteria, after reviewing the AET in 1989, recommended that multiple approaches be used to estimate sediment quality and that regionally developed SQC are inappropriate for nationwide usage.

For ecological assessment and regulation of dredged sediment, the present regulatory approach is deemed adequate. It is an effects-based approach that fully considers interactive and antagonistic effects of contaminants associated with sediments. Regardless of whether SQC are implemented, the Corps' management strategy will still be required to make informed decisions on disposal options and controls for dredged material disposal.

## ACKNOWLEDGEMENTS

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# Table 1

## Comprehensive Testing Approach for Aquatic Disposal
### as Part of the Federal Standard*

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**Tier 1**   Initial evaluation of existing information and "reason to believe" there is contamination.

**Tier IIA**   Bulk sediment inventory. Reason to believe dredged material is more contaminated than disposal site sediment and potential unacceptable adverse effects may occur.

**Tier IIB**   Elutriate analysis. Chemical analysis for contaminant(s) of concern, contrast to appropriate water-quality criteria and/or standard with consideration of mixing. Comparison to receiving water quality and/or bioassay when no standard exists.

**Tier III**   Biological tests.

**Tier IIIA**   Acute bioassay toxicity tests (as appropriate):

| Water Column (Elutriate) | Select Species |
|---|---|
| (Mixing considered) | (As necessary) |
| Dissolved phase | Mysid shrimp |
| Suspended solids phase | Grass shrimp |
| | Bivalve |
| | Fish |
| | Larva, bivalve |
| | Other |
| | |
| Benthic | |
| Solid phase | Mysid shrimp |
| | Amphipod |
| | Grass shrimp |
| | Clam |
| | Polychaete |
| | Other |

**Tier IIIB**   Bioaccumulation.

| Water Column | Select species |
|---|---|
| Suspended solids phase | Grass shrimp |
| | Clam |
| | Polychaete |
| | Other |
| Benthic | |
| Solid phase | Clam |
| | Polychaete |
| | Other |

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* Table 1 (from Engler, et al. 1988) presents the general types of tests and evaluations in a tiered and sequential basis where each tier (step) is optional and may be eliminated or chosen as appropriate. Test species tested are not mandatory but are shown for consideration to a proposed disposal site region.the same as those presented earlier in the discussion of sediment bioassays.

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RESERVOIR AND RIVERINE SESSION

# MANAGING LONG TERM RESERVOIR WATER QUALITY

by

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## INTRODUCTION

Numerous Corps reservoirs suffer from a multitude of problems including shoreline erosion, sedimentation, degraded water quality and lost or diminished fisheries. In addition, urbanization often results in erosion problems as well as degraded water quality. Avoidance or minimization of these problems requires recognition, expertise and most importantly, early awareness on the part of management. All of the above elements are required to effectively manage and prolong the life of our reservoirs.

Dam building agencies traditionally have not designed reservoirs, they have designed dams. Nearly the entire effort has gone into the design of the dam with little consideration given to the reservoir except as a volume to be filled with water. While the importance of safe dams cannot be overemphasized, the philosophy of considering the reservoir only as a volume to be filled with water has resulted in a multitude of problems which could have been minimized or avoided.

Reservoir management tends to be piecemeal. One agency manages the fishery, another agency or part of an agency manages grazing leases, another part of an agency looks at camp pads or grass mowing, another part of an agency manages the releases, still another part of an agency works with water quality. Even if all the management parts perform their function, the reservoir still suffers. Rarely, if ever, is the reservoir considered as an ecological whole and will not live up to its useful potential if managed as a group of unrelated parts. Because of this piecemeal management, our reservoirs and their recreation functions are being lost or degraded. It is imperative that we work with other agencies and consider the reservoirs along with their watershed as an ecological whole. We can no longer afford to stop at the project boundaries as a routine part of our job nor can we afford to treat our reservoirs as a group of unrelated parts. In order to maintain aesthetically appealing, functional, and high quality water-based recreation we must manage our reservoirs as a total ecosystem.

The following sections discuss some major reservoir problems and provide some corrective and preventative solutions.

## URBANIZATION

Urbanization is a major problems facing many of our reservoir projects today. Unfortunately, such problems often arise quickly and are thus dealt with on a crisis management basis. Under a crisis management scenario, once a problem is handled in a particular manner a precedent is often set which can make future changes in handling similar problems difficult. Urbanization problems associated with water quality are basically of two types; those associated with the construction period and those associated with the post-construction period.

Construction problems are commonly associated with laying the land bare of vegetation thus allowing sediment laden run-off to reach nearby reservoirs or waterways. Obviously there are methods of minimizing this type of impact; temporary sediment ponds, staging construction so large areas are not denuded, using bales or sediment curtains, and using terraces as sediment interceptors. However, construction site visitation (both Corps and private) indicates that the use of such methods is not common. It would benefit our projects to require Corps contractors and project sponsors to utilize

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construction methods which minimize sediment problems. It would also benefit the projects if legal actions were taken or threatened against construction activities damaging to the reservoir.

Post-construction problems are commonly associated with storm drainage. Storm sewers not only short circuit urban pollution to the stream or reservoir, they commonly exit on or near project lands and can result in severe erosion problems. Obviously the conversion of grasslands or agricultural lands to water impervious surfaces such as roads, roof-tops and sidewalks increases both the flow rate and the volume of water draining from a given watershed. Some states require detention be provided to ensure stormwater discharges originating from a developed drainage do not exceed the 100-year historic flows under undeveloped conditions. However, it must be remembered that 100-year flows are catastrophic to most natural drainages and that urbanization increases the frequency of catastrophic events.

A variety of measures are available to minimize the impacts of post construction urbanization. Urban storm flows that are detrimental to receiving waters can be permitted under the National Pollution Discharge Elimination System (NPDES) thus requiring treatment. Storm sewer exits can be allowed on project lands provided the developer will go to some effort to provide detention in the form of ponds swales or wetlands on private property. In addition, the developer might be asked to construct a series of wetlands to slow downhill flows and provide time for bacterial die-off, chemical degradation, and reduced flow rates. The advantage to the developer would be the construction of the storm sewer outlet on project property which may allow for an extra structure or more aesthetic conditions. (It is important to note that "wetland" as defined for purposes of this paper is a shallow basin not more than 1-2 feet in depth and filled with aquatic plants. It should not be defined as a pond which can, under the right circumstances can become an attractive nuisance.)

There are numerous viable methods of minimizing urbanization impacts, however, these methods are rarely considered because of piecemeal management. Very commonly, those reviewing drainage plans are not familiar with the environmental problems associated with drainage. Obviously, these groups should work together, not independently.

It is suggested that an organized process or policy be created to handle urbanization problems in a reasonable and consistent manner. Such a policy should involve water quality, natural resources management, engineering expertise and should work with developers to reach an amiable agreement as opposed to simply accepting whatever damages which might occur. A variety of scenarios could be developed, their primary purpose being to maintain the quality and longetivity of the project and good relations with the development interests. Such actions can result in benefits to both parties.


## SHORELINE EROSION

Shoreline erosion is one of the most ubiquitous problems facing our reservoirs today. The Omaha District has nearly six thousand miles of shoreline on six mainstem reservoirs (Corps of Engineers, 1988) with additional miles of shoreline erosion occurring at all tributary projects. While the precise quantity of eroding shoreline has not been ascertained, it can be stated that between 70 and 90% of the shoreline is eroding. Shoreline erosion can result in a decrease in overall water quality because of the addition of sediment and associated nutrients, turbidity increases which in turn can limit light thus limiting aquatic vegetation important biota including fisheries, the necessity to purchase additional private properties and the loss of physical structures such as recreation facilities. Unfortunately, shoreline erosion is commonly ignored since it is often not perceived as a problem to the observer unless structures or private property are threatened.

Historically the primary solution to shoreline erosion problems has been the use of riprap. While riprap can solve the problem, it is often not the best ecological solution. Mile after mile of rock shoreline does not provide the diversity required to maintain a stable ecosystem. In addition, riprap can be extremely expensive which can result in ignoring the problem based simply on costs. Obviously, ignoring the problem allows shoreline erosion to continue unabated.

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Aquatic plants provide another alternative to managing shoreline erosion problems. Shoreline stabilized with vegetation can provide for greater ecological diversity even coupled with riprap, thus resulting in a more stable lake system ecosystem. Aquatic plants will stabilize some shoreline areas naturally provided that water level fluctuation and other erosive forces are not great enough to prevent their establishment. Natural stabilization however can take many years and can result in large quantities of sediment input to the reservoir system.

The Waterways Experiment Station has developed an excellent program which utilizes aquatic plants and can provide technical expertise in resolving shoreline erosion problems. Their methods are often cheaper than riprap and a living system can maintain itself and spread to untreated areas if conditions are appropriate. In addition, an aquatic vegetation program can be a budgetary line item or initiated with one or two thousand dollars squeezed out of an existing budget. Shoreline erosion treated on a piecemeal basis, even a few feet of shoreline each year is preferable to doing nothing.

It is obvious that a policy should be established requiring the identification and treatment of some portion of eroding shoreline at each project. This process could be a budgetary line item or a project handled on an "as time permits" basis. It is imperative that we begin resolving an all too common problem in order to protect and preserve our reservoir resources.

## SEDIMENTATION

In the agricultural midwest, as well as other parts of the nation, sedimentation is one of the major problems leading to the degradation of reservoirs in terms of water quality, recreation, and reservoir longevity. A case in point is the loss of the bass-bluegill fishery in a group of ten Salt Valley reservoirs located near Lincoln, Nebraska (Schainost, 1989). Imagine, a major part of the recreation lost in less than 20-25 years, a very small time increment in terms of the life of the reservoir. It is widely recognized that sedimentation will impact reservoirs and EM 1110-2-4000 states that sedimentation rates should be determined during the planning process. However, sedimentation problems will impact fisheries, water quality, and other reservoir functions years before any impact to flood control occurs.

As a remedy to sedimentation problems extant in many parts of the country, methods of minimizing the problem to extend the usability and longevity of the reservoir should be implemented in both the planning and operational phases. We should cooperate with agencies such as the Soil Conservation Service (SCS) which has the authority to work with erosion problems outside of project boundaries. We should cooperate with states to develop and implement Clean Lakes Programs or Non-Point Source Programs, even though such programs may not relate specifically to a Corps project. At the same time, we should not be misled into believing that our projects are safe from sedimentation because of terraces, grassed waterways, or a high percentage of treated land exists in the watershed. Terraces for instance are normally designed to provide protection only up to the 10-year storm event (U.S. Department of Agriculture 1988). While terraces and grassed waterways are unquestioningly valuable, we cannot be lulled into believing that our projects are free from sedimentation problems.

Sediment detention, in the form of wetlands or ponds should be considered during the planning process as should the use of wetlands as water quality improvement. Research has repeatedly demonstrated that wetlands can function to improve water quality (Whigham, et. al. 1988, Knight, et. al. 1987). Wetlands as water quality filters can be constructed simultaneously with the embankment and can be constructed either as an integral part of the reservoir or totally separate from the reservoir (upstream of the reservoir). The area in which a delta will form can easily and cheaply be converted to wetlands or sediment entrapment structures. While the trapping efficiency of a structure may be questioned, there is little doubt that sediment will be trapped, bacterial die-off time will be increased, and some nutrients and pesticides will fall out with the sediment. In addition, such traps can be designed for easy clean-out to maintain their usefulness. In short, such measures can be devised and provide an increased measure of protection for valuable reservoir projects. Managing the problem of dredging a sediment trap is cheaper than dredging an entire project. It is important that the problem be recognized and dealt with.

## SUMMARY

At present, our reservoirs are being lost and their usefulness limited due to shoreline erosion, sedimentation, and urbanization. It is imperative that we recognize our reservoirs as important resources and that we work together to correct problems and manage them as ecological systems rather than the sum of their parts. It is also imperative that we plan and build quality reservoirs as well as safe dams. This is the only manner in which we can preserve their usefulness for ourselves and future generations.

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# WATER QUALITY MANAGEMENT FOR RESERVOIRS AND TAILWATERS:
## AN OVERVIEW

by

Robert H. Kennedy[1]

## INTRODUCTION

The Water Quality Management for Reservoirs and Tailwaters (WQMRT) Demonstration, as part of the Water Operations Technical Support (WOTS) Program, is designed to provide the informational base from which Corps of Engineers (CE) personnel can develop and implement sound water quality management initiatives. Specific objectives being addressed within WQMRT are to:

    a. Identify water quality enhancement needs

    b. Inventory existing management technologies

    c. Develop guidelines for establishing and implementing
       water quality management plans

Several approaches were employed to meet these objectives. Data describing water quality conditions at CE water resource projects were retrieved from selected computer databases, principally STORET (US Environmental Protection Agency, 1979). Additionally, field operating activity (FOA) evaluations of conditions at each project were solicited using a comprehensive questionnaire. Together, these two types of information allowed description of CE-wide patterns in water quality conditions and enhancement needs. Preliminary results of attempts to identify water quality enhancement needs at CE water resource projects were reported by Kennedy et al. (1988). Gaugush and Kennedy (1990) provide a discussion of interrelations between water quality data and subjective evaluations of conditions in selected CE divisions.

Information concerning technologies applicable to FOA water quality management activities was gathered from a variety of sources and compiled for ease of use by project personnel. Many of the techniques were originally developed for and applied in natural lakes; however, many approaches for dealing with water quality management issues are applicable for reservoirs as well. Techniques involving structural or operational modifications received greatest attention. FOA responses to the questionnaire provided information concerning recent water quality management experiences at CE projects. Applicable in-reservoir water quality management techniques are compiled and discussed in Cooke and Kennedy (1989), while operational and structural techniques are discussed in Price (in prep). The report by Cooke and Kennedy also includes a discussion of useful techniques for problem assessment and diagnosis.

Combining accurate problem diagnosis and assessment with potential problem solutions is obviously the key to sound water quality management. Engineering Manual EM 1110-2-1201, "Reservoir Water Quality Analyses," describes a wide range of techniques for data analysis and overall assessment of water quality. These include guidelines for sample design, statistical and graphical methods for data evaluation, and the use of indices and models. Since the preparation of this manual, a number of new techniques and refinements of established techniques have been reported. Therefore, the manual will be updated to insure that the latest techniques are available to FOA personnel. At the same time, new information concerning water quality management will be incorporated, thus expanding the scope of the manual and increasing its value to water quality managers.

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Demonstrating the utility of techniques and approaches will be an important part of the WQMRT. Cooperative efforts by the US Army Engineer Waterways Experiment Station (WES) and the US Army Engineer District, Baltimore (NAB), provide an example of a technically-sound approach to water quality management at a small, flood-control project (Ashby et al., 1990). Water quality concerns at East Sidney Lake, New York, have been addressed by coupling watershed management concepts with in-reservoir management techniques to provide an holistic approach to water quality management.

## WATER QUALITY MANAGEMENT TECHNIQUES

Almost all water quality concerns in reservoirs and their tailwaters are related, directly or indirectly, to the process of eutrophication. Eutrophication is the process by which algal growth-stimulating nutrients, oxygen-demanding organic material, and sediment accumulate in lakes and reservoirs. Reduced water clarity, excessive growths of algae and other aquatic plants, exhausted oxygen supplies in bottom waters, and impacted fisheries are some of the consequences of this process. To reduce the severity of these and other consequences, water quality management efforts must (1) reduce the quantity of unwanted materials transported from watershed to reservoir, (2) reduce the accumulation or availability of materials which do reach the reservoir, and/or (3) control symptoms of eutrophication directly.

Curtailing the influx of nutrients and sediment is obviously the best water quality management method. However, regulation of watershed activities and control of waste discharges are not within the CE mission and the cooperation of responsible agencies must be sought. General approaches for reducing material loads to reservoirs include treatment of point sources and institution of best management practices (BMPs) to curb nonpoint source material losses. For projects with significant project lands, every effort should be made to reduce the loss of materials to the reservoir. This can be particularly important for projects with isolated coves and embayments which may be more impacted by local runoff events than by inflows from the major tributaries.

Considering the difficulties inherent in controlling inputs to reservoirs, CE water quality personnel will most often be faced with managing water quality within the project or through operational or structural modifications. Table 1 provides an overview of the wide range of techniques applicable for reservoirs. While some techniques are specific in their application and limited in their mode of action, others can be used to accomplish multiple objectives. For instance, aeration of hypolimnetic waters devoid of oxygen during the summer months will improve tailwater dissolved oxygen concentrations, as well as reduce the rate at which nutrients and metals are released from bottom sediments. Selection of appropriate water quality management alternatives will require careful consideration of factors influencing water quality, current water quality conditions, and expected technique impacts and effectiveness.

Reports of FOA applications of these techniques at CE projects (Price, in preparation) indicate that several techniques have been employed at CE projects with varying degrees of success (Tables 2 and 3). The most frequently cited, and apparently most successful, are operational modifications. Water quality concerns addressed by this method are centered mostly on the tailwater area and include low dissolved oxygen concentration, undesirable temperature regime, and elevated metal concentrations. While methods to reduce inputs to projects from the watershed and shoreline areas have been attempted, the application of a number of in-reservoir methods have also been reported. Unfortunately, post-implementation evaluations were not available for many projects.

## A CASE STUDY

Combined WES-NAB activities at East Sidney Lake, a small flood-control project located in south-central New York, exemplify the holistic approach to water quality management. Efforts here involved delineation of watershed activities, coordination with local agencies, surveys of use patterns and user perceptions of water quality, and implementation of an in-reservoir water quality management technique (Ashby et al., 1990).

**Table 1. General water quality management techniques applicable to reservoirs and tailwaters. Based on Cooke and Kennedy (1989) and Price (in prep).**

Watershed Techniques:

| | |
|---|---|
| Pre-reservoir treatment | Best management practices |
| Shoreline erosion control | |

In-reservoir Techniques:

| | |
|---|---|
| Phosphorus inactivation | Dilution and flushing |
| Sediment removal | Hypolimnetic aeration |
| Artificial circulation | Water level drawdown |
| Destratification | Oxygenation |
| Harvesting | Biological controls |
| Sediment covers | Herbicides and algicides |

Operational/Structural Techniques:

| | |
|---|---|
| Rule curve modification | Inflow routing |
| Supplemental releases | Concentration of release flow |
| Release strategy optimization | Hypolimnetic withdrawal |
| Submerged weirs | Turbine venting |
| Selective withdrawal | |

Problem assessment followed delineation of watershed landuse patterns through interpretation of aerial, infrared photographs, estimation of nutrient loads based on routine sampling of tributaries, and monitoring of seasonal changes in pool water quality. Clear from this assessment is the fact that agricultural activities in this 264-km$^2$ watershed result in excessive loads of both nitrogen and phosphorus reaching the lake (Kennedy et al., 1988; Ashby et al., 1990). In response, algal blooms often occur during the summer months, creating scums and reducing water clarity. Although only weakly stratified during summer months, the lake becomes devoid of dissolved oxygen in bottom waters and large quantities of soluble phosphorus are subsequently released from sediments, exacerbating problems of excessive algal growth. A survey of lake users indicated that conditions in mid to late summer were often less than desirable.

To break this cycle of nutrient loading and internal recycling, a two-part water quality management plan was designed. First, a pneumatic destratification system was placed in deep water in the lower end of the lake to prevent anoxia during the stratified period and to reduce algal abundance through reduced internal loading. While the performance of this system is still being evaluated, initial results are encouraging (Ashby et al., 1990).

Second, NAB has coordinated with state and local agencies whose responsibility it is to encourage local residents to modify practices and activities which lead to excessive nutrient loads. Data

**Table 2. Water quality enhancement techniques recently employed at CE projects. Based on Price (in prep).**

| Technique Applied | Degree of Success | | |
| --- | --- | --- | --- |
| | Successful | Marginal | Unknown |
| Operational modification | 26 | 1 | 2 |
| Structural modification | 0 | 3 | 1 |
| Selective withdrawal | 3 | 3 | 4 |
| Localized mixing | 1 | 0 | 0 |
| Drawdown and planting | 1 | 0 | 11 |
| Hypolimnetic withdrawal | 0 | 0 | 1 |
| Inflow diversion | 1 | 0 | 0 |
| Destratification | 1 | 1 | 0 |
| Algicide and herbicide | 0 | 5 | 2 |
| Wetland creation | 0 | 0 | 1 |
| Dredging | 0 | 1 | 0 |
| Best management practice | 0 | 0 | 1 |
| Pool fluctuation | 0 | 1 | 0 |
| Liming for pH control | 2 | 0 | 0 |
| Grass carp | 0 | 0 | 1 |

**Table 3. CE-wide distribution of recent water quality management initiatives. Based on Price (in prep).**

| CE Division | Number of Projects |
| --- | --- |
| Lower Mississippi Valley | 8 |
| Missouri River | 23 |
| North Atlantic | 12 |
| North Central | 1 |
| New England | 3 |
| North Pacific | 19 |
| Ohio River | 18 |
| South Pacific | 1 |
| Southwest | 12 |

collected by the CE are also being used to support efforts to obtain funding for nonpoint source control under the US Department of Agriculture's Water Quality Plan. Additionally, study results have been publicized as an effort to raise public awareness of ongoing efforts (Bank, 1989).

## SUMMARY

As the demand for water-based recreation and the expectations of project users for "good water quality" increase in the coming years, the CE must be prepared to take a leading role in the preservation of the water resources within its control. Information compiled during the WQMRT Demonstration and the establishment of guidelines based on this information will provide FOA water quality personnel with means for developing technically-sound management plans.

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# RELATIONSHIP BETWEEN OBSERVED AND PERCEIVED WATER QUALITY IN CORPS OF ENGINEERS RESERVOIRS

by

Robert F. Gaugush and Robert H. Kennedy[1]

## INTRODUCTION

In 1987, the Water Quality Management for Reservoirs and Tailwaters (WQMRT) Demonstration was added to the Water Operations Technical Support (WOTS) Program. The objectives of the demonstration, sponsored by the Office, Chief of Engineers, are to:

a. Identify water quality enhancement needs;
b. Inventory existing management technologies;
c. Develop a protocol for establishing and implementing water quality management plans.

The first objective is being met through the use of a comprehensive questionnaire and by developing a Corps of Engineers (CE) reservoir database. The questionnaire solicited detailed information about project operation and was designed to allow district personnel to subjectively grade each project's water quality attributes. A preliminary discussion of the identification of water quality enhancement needs based on the questionnaire was presented by Kennedy et al. (1988). The reservoir database was constructed by retrieving water quality information from STORET (U.S. Environmental Protection Agency, 1979).

One task under the objective of identifying water quality enhancement needs is concerned with the description of the regional component of the variability in water quality. A consideration of regional differences in water quality will be of prime importance in the development of attainable water quality goals. Water quality goals cannot be developed on a national scale without addressing regional differences in hydrology, nutrient loading, and climatic conditions.

An initial step in developing regional descriptions of attainable water quality is to examine the relationship between perceived water quality and measured or observed water quality. Regional aspects of both perceived and observed water quality will determine the nature and extent of water quality problems and enhancement needs.

### OBSERVED AND PERCEIVED TROPHIC STATE

As part of the questionnaire, district personnel were asked to classify the trophic state of the reservoir pool. It was recognized that although numerous criteria and indices have been used to describe trophic state in objective terms (such as nutrient or chlorophyll concentrations), many regional differences are apparent in individual perceptions of trophic state. The subjective definitions of trophic state presented in Table 1 were used by the respondents to describe the local perception of trophic state.

For the purposes of this paper, four divisions were chosen to examine the relationship between observed and perceived trophic state. The New England (NED), North Atlantic (NAD), South Atlantic (SAD), and North Pacific (NPD) Divisions were selected to allow for regional differences in the perception of trophic state. Questionnaire data for these divisions were merged with "hard" data retrieved from STORET and this combined data set was analyzed to determine if regional conditions influenced the perception of trophic state.

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Table 1. Subjective definitions used for the classification of trophic state.

| Trophic State | Definition |
| --- | --- |
| Oligotrophic | Low nutrient and chlorophyll concentrations, high water clarity, and/or unimpaired dissolved oxygen conditions in bottom waters |
| Mesotrophic | Intermediate nutrient and chlorophyll concentrations, reduced water clarity, and/or moderately impaired dissolved oxygen conditions in bottom waters |
| Eutrophic | High nutrient and chlorophyll concentrations, greatly reduced water clarity, and/or highly impaired dissolved oxygen conditions in bottom waters |
| Hypereutrophic | Extremely high nutrient and chlorophyll concentrations, minimal water clarity, and/or severely impaired dissolved oxygen conditions in bottom waters |
| Undetermined | Insufficient information available to make such an evaluation |

In terms of perceived trophic state these four divisions are very different (Figure 1). NAD is characterized by a fairly even distribution of trophic state. In NED, the majority of the reservoirs was classified as mesotrophic with a secondary cluster of oligotrophic reservoirs. Most reservoirs in NPD were reported as oligotrophic with mesotrophic reservoirs comprising most of the remainder. In SAD, on the other hand, most reservoirs are considered eutrophic with a smaller number classified as mesotrophic. Relatively few projects were reported with an undetermined trophic state.

Chlorophyll concentration, water clarity, and nutrient concentrations were used in a subjective sense in the questionnaire to specify the perceived trophic state. These same variables can be used in an objective sense to examine the relationship between observed and perceived trophic state. An analysis of variance was used to determine whether or not perceived trophic state was associated with consistent differences in common water quality indicators (chlorophyll $a$, Secchi depth, total phosphorus, and total nitrogen) of trophic state (Figure 2).

Based on the data derived from these four divisions, an oligotrophic reservoir cannot be distinguished from a mesotrophic reservoir for any of the variables considered. Secchi depth and total nitrogen concentration share a similar pattern with respect to reported trophic state. Significant differences exist between a less enriched group (oligotrophic and mesotrophic reservoirs) and a more enriched group (eutrophic and hypereutrophic reservoirs). For chlorophyll $a$, the most common indicator of trophic state, and total phosphorus, the most common determinant of trophic state, eutrophic and hypereutrophic reservoirs are readily distinguished from each other and from less enriched systems. Chlorophyll $a$, total phosphorus, and total nitrogen concentrations in reservoirs reported as hypereutrophic are highly variable as indicated by the size of the error bars associated with these variables.
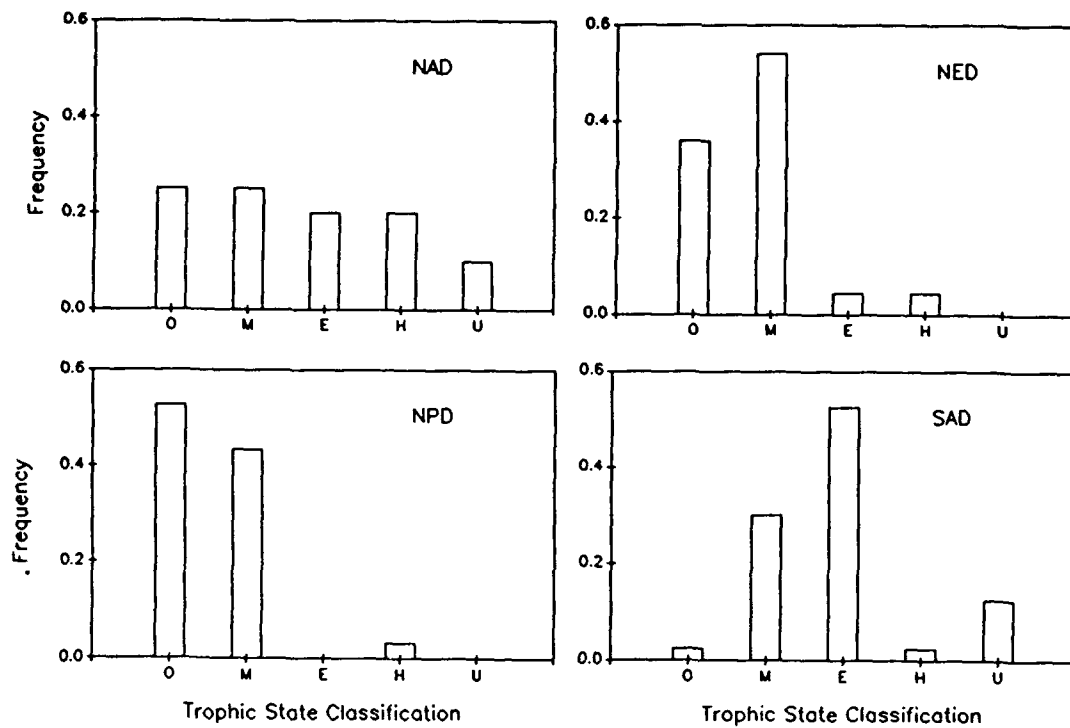
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Figure 1. Trophic state classifications for NAD, NED, NPD, and SAD. Reported trophic state indicated by letter: O - oligotrophic, M - mesotrophic, E - eutrophic, H - hypereutrophic, U - undetermined.
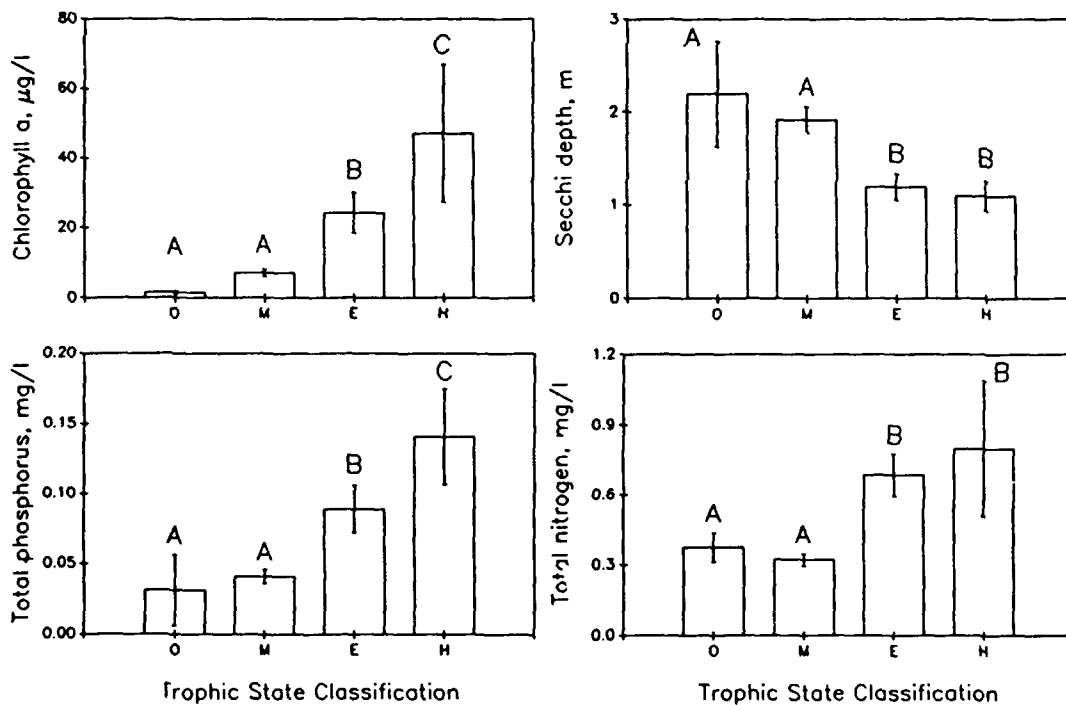


Figure 2. Mean chlorophyll a, Secchi depth, total phosphorus, and total nitrogen as related to reported trophic state. Means marked with the same letter are not significantly different.

The lack of a significant difference between measured values for reservoirs reported as oligotrophic and mesotrophic suggest that there are regional aspects in the perception of these two trophic states. Also, a regional component may explain the high variability observed within reservoirs reported as hypereutrophic. A second analysis of variance was conducted to test for significant regional differences by analyzing each reported trophic state with respect to CE divisions. Using chlorophyll a concentrations, significant regional differences with respect to reported trophic state were found (Figure 3). For oligotrophic reservoirs, chlorophyll concentrations are all well below 10 $\mu$g/l, but concentrations in NED are significantly higher than those in either NAD or NPD. In mesotrophic reservoirs, two very different groups can be identified. The first group, reservoirs from NAD and SAD, has chlorophyll a concentrations approaching 10 $\mu$g/l, while the second group, reservoirs from NED and NPD, has chlorophyll a concentrations less than 3 $\mu$g/l. The regional differences observed for oligotrophic and mesotrophic reservoirs may be sufficient to explain the lack of significant differences between these two trophic states when regions (or CE divisions) were not considered. Hypereutrophic systems in NAD and NED are significantly different which may explain the high degree of variability observed for hypereutrophic reservoirs as a group.

## OBSERVED AND PERCEIVED WATER QUALITY ENHANCEMENT NEEDS

In addition to the classification of trophic state, district personnel were asked to evaluate the water quality conditions of their projects. For a number of commonly cited water quality considerations (such as low dissolved oxygen, high nutrient concentrations, excessive algal biomass, etc.), respondents were asked to evaluate the extent of the problem using the problem levels presented in Table 2. Analysis of the questionnaire has indicated that the most prevalent problems are those associated with the eutrophication process (Kennedy, et al., 1988). These included excessive nutrient concentrations, algal blooms, and anoxic conditions in the hypolimnion.

One of the most commonly cited water quality problems was "algae - excessive algal biomass or chlorophyll concentration." For a preliminary analysis of the relationship between the reported problem level and observed water quality data, data from four divisions (NAD, NED, NPD, and SAD) were selected. Reported problems associated with the algal category for these divisions are presented in Figure 4. Most reservoirs were reported to have no problem in the area of algal density, but a considerable fraction was reported to have intermittent or occasional problems. Very few reservoirs were reported to have chronic problems with excessive algal biomass.

In order to examine how well reported problems agreed with observed water quality, an analysis of variance was conducted using chlorophyll a, Secchi depth, total phosphorus, and total nitrogen as variables associated with algal problems. Results of the analysis of variance are reported in Figure 5. Concentrations of chlorophyll a exhibit a pattern that is consistent with the reported level of the problem. Reservoirs with chronic problems have significantly higher chlorophyll concentrations than those with intermittent or occasional problems. Chlorophyll levels in reservoirs with either occasional or no problems cannot be distinguished. Total phosphorus, in most cases the limiting factor for algal production, exhibits a similar pattern. Secchi depth and total nitrogen concentrations do not display a consistent pattern with respect to the reported problem levels.

In order to identify regional aspects in the perception of algal problems a second analysis of variance was conducted with respect to CE division (or region). SAD has significantly higher chlorophyll a concentrations in the intermittent, occasional, and no problem categories (Figure 6). This implies that much higher chlorophyll a concentrations can be attained in SAD before a problem is perceived to exist. Chlorophyll a concentrations for SAD in the no-problem category approach 10 $\mu$g/l. These same concentrations would be classified as an intermittent problem in the other divisions. In SAD where nutrient loading is relatively high and climatic conditions are favorable for algal production, 10 $\mu$g/l of chlorophyll a may represent baseline conditions.
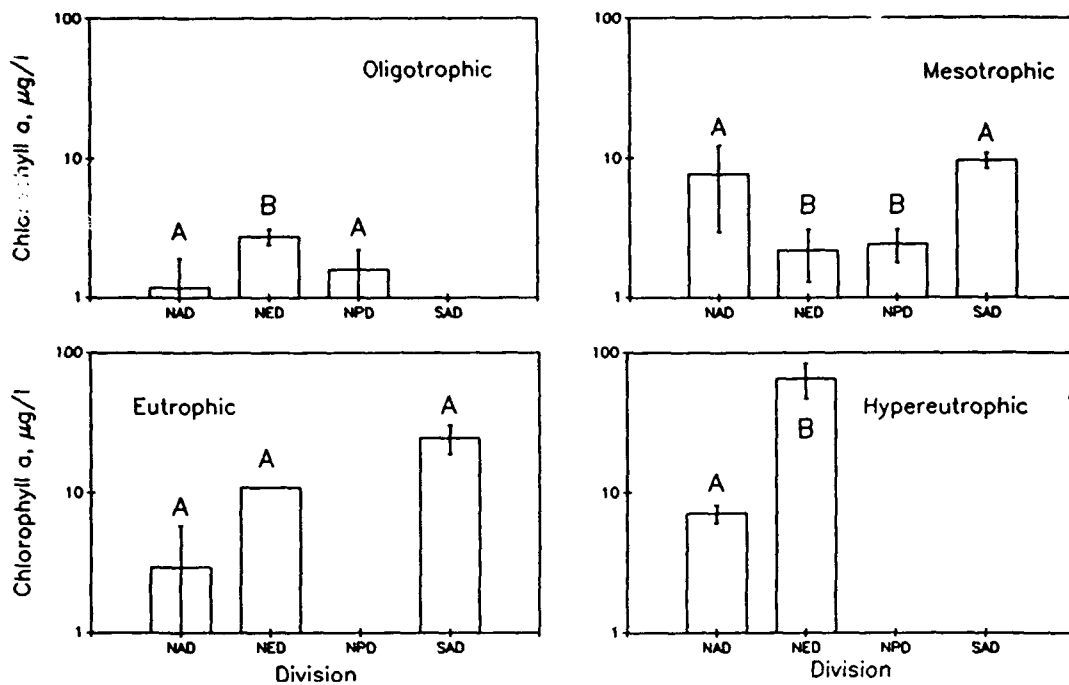
Figure 3. Mean chlorophyll a concentrations by Division for reservoirs with a reported trophic state of oligotrophic, mesotrophic, eutrophic and hypereutrophic. Means marked with the same letter are not significantly different.
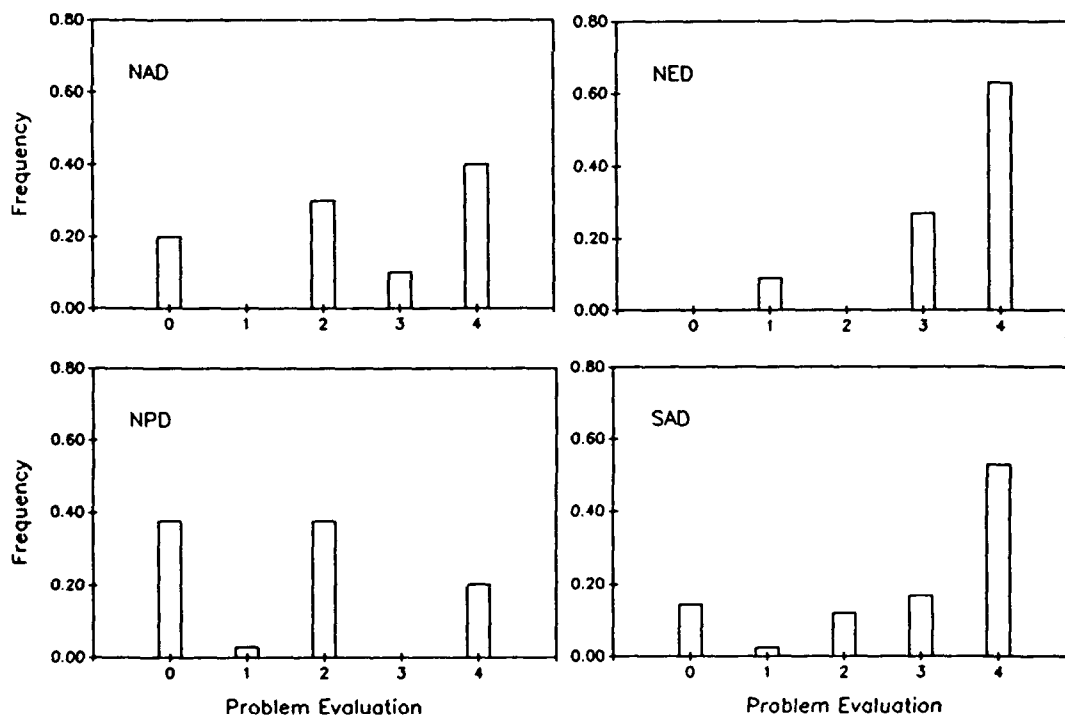


Figure 4. Evaluation of algal problems for NAD, NED, NPD, and SAD. Reported level of the problem indicated by number as given in Table 2.
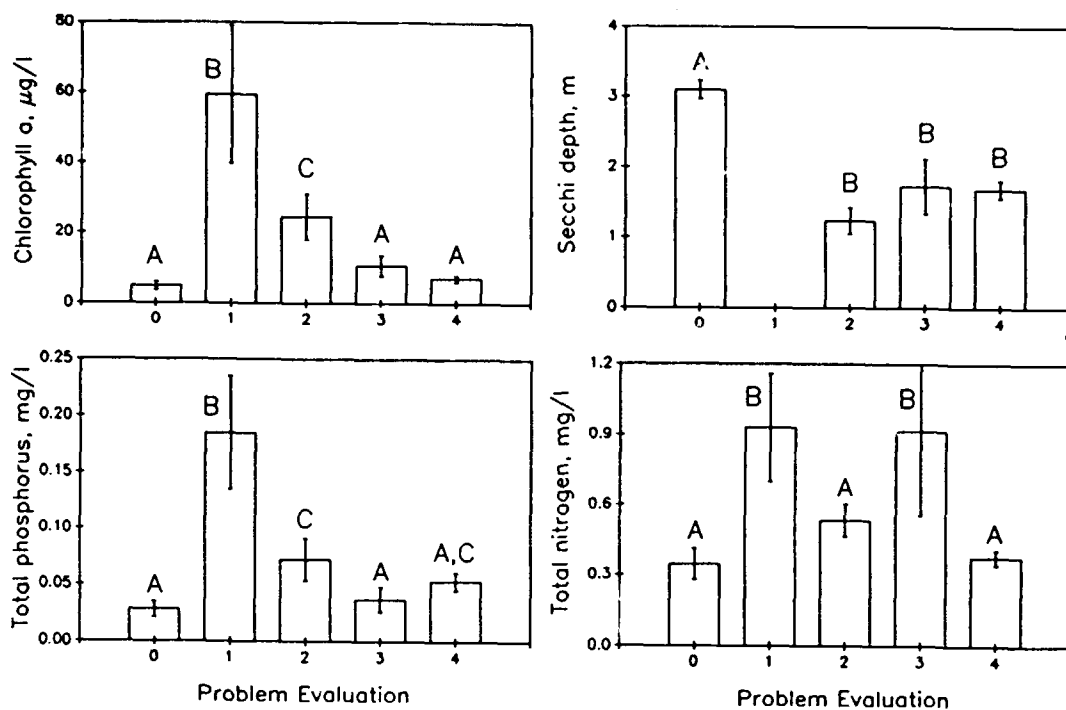
Figure 5. Mean chlorophyll a, Secchi depth, total phosphorus, and total nitrogen as related to reported problem level. Means marked with the same letter are not significantly different.
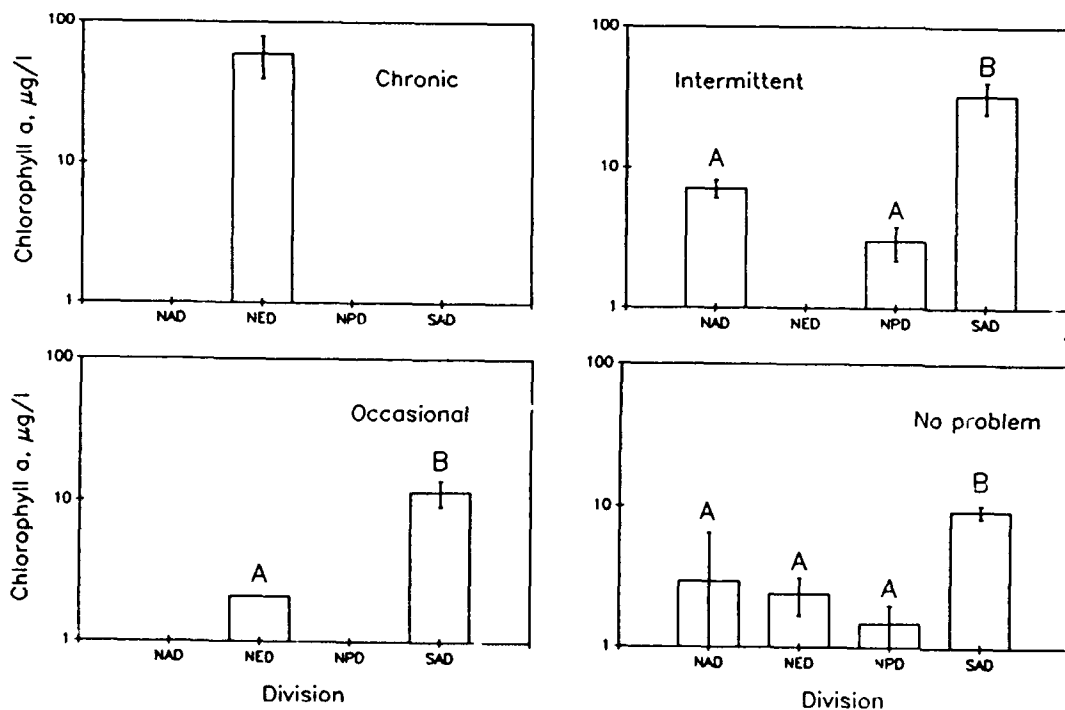


Figure 6. Mean chlorophyll a concentrations by Division for reservoirs with a reported problem level of chronic, intermittent, occasional, and no problem. Means marked with the same letter are not significantly different.

Table 2. Definitions used to specify the extent of water quality problems.

| Problem Level | Definition |
| --- | --- |
| 0 | No problem evaluation has been made |
| 1 | Chronic or continuous problem |
| 2 | Intermittent problem occurring on a seasonal or event basis |
| 3 | Occasional problem occurring infrequently on an annual basis |
| 4 | No problem |

## CONCLUSIONS

Regional differences in both the perception of trophic state and its measurement are apparent. Regional differences in the perception of water quality problems are also readily apparent. These differences suggest that, in reservoirs, trophic state and water quality are evaluated or judged with some regional bias. This bias may be a result of the perception of ambient water quality in natural lakes within the region. For example, in a region like the North Pacific Division with relatively numerous lakes with very good water quality, reservoirs may be judged with respect to those lakes rather than with respect to the reservoirs in the division. In the South Atlantic Division, on the other hand, where natural lakes are relatively scarce, reservoir water quality will most likely be judged with respect to other reservoirs. The regional aspects of trophic state and water quality problem evaluation must be addressed in the establishment of reasonable and attainable water quality goals.

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# WILLOW CREEK LAKE WATER QUALITY STUDIES

by

Richard A. Cassidy[1]

## INTRODUCTION

Willow Creek Dam is a U.S. Army Corps of Engineers multiple-purpose project located on a tributary of the Columbia River (Willow Creek) in north-central Oregon. Completed in 1983, it was the world's first dam entirely built of roller compacted concrete (RCC). The 1,780-foot-long dam is 169 feet high, and has a 384-foot-wide uncontrolled gravity spillway section located in the central portion of the structure. A 970-foot-long drainage gallery extends through the gallery, sloping downward from the left bank to the right bank. There is a 60-foot-diameter gated regulating outlet tunnel capable of discharging 500 cubic feet per second ($ft^3/s$). There is also a telescoping selective withdrawal outlet structure capable of releasing up to 80 $ft^3/s$.

## GEOCHEMICAL AND BACTERIOLOGICAL

Water quality analyses were conducted between 1986 and 1989 to determine whether dissolution of concrete within the dam was affecting dam safety and whether $H_2S$ was a health hazard. An analytical program was first designed to identify the geochemical and bacteriological processes which appeared to accompany the leakage of water through the dam structure. By 1988, refined sampling, processing, and analyses of reservoir and dam seepage water were conducted by contractors from Oregon State University and the University of Washington, to better define the rates and processes that were occurring.

As a result of these geochemical and bacteriological studies, media attention in 1988 prompted Senator Mark O. Hatfield and Robert W. Page, Assistant Secretary of the Army for Civil Works, to request a special investigative team to evaluate the project. They concluded that Willow Creek Dam was safe and represented no health hazards. The team recommended completion of 11 geotechnical studies and investigations already underway. Those studies completed in January 1990 support the investigative team's conclusion that the dam is safe and presents no health hazard from $H_2S$.

## SEEPAGE

During reservoir filling in the fall of 1983, water began seeping through the dam at approximately 3,000 gallons per minute (gpm). Filling was temporarily stopped so a grouting program could be implemented. Portland cement grout was injected into vertical holes drilled along most of the length of the dam. Seepage was reduced to 90 gpm at the low pool level. When the impoundment was filled to its normal operating pool level, with 90 feet of head in February 1984, the seepage rate was approximately 900 gpm. Seepage reduced to about 500 gpm by early 1985; however, calcite deposition became evident in the gallery and on the downstream face of the dam during 1984. Calcite continued to build during 1985 and, in addition, $H_2S$ was detectable by smell in the gallery. Seepage has continued to decrease to about 90 gpm as of January 1990.

## CORROSION

Corrosion of a concrete structure typically starts with the dissolution of the free lime initially hydrated during the hardening of the concrete (Biczok, 1967). Other uncombined lime (CaO) will also be hydrolyzed. Calcium silicates and aluminum phases will continue to hydrolyze slowly as long as percolation occurs. Leaching of lime increases the permeability of concrete; however, the loss in lime must be substantial before the strength is seriously affected. Lea (1970) suggested, as a rough approximation, that 1 to 2 percent of the cement strength of portland cement is lost for each percent

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loss in lime. Much of the $Ca(OH)_2$ reacts with the fly ash added to the concrete, therefore, the portland-pozzolan cements used in the RCC are relatively resistant to this type of corrosion. The grouting pumped into the dam can be expected to contain a significant higher percent of free lime.

Under some conditions, dissolved carbon dioxide as $H_2CO_3$ in the percolation watcon can act as an acid which accelerates dissolution:

$$2H_2CO_3 + CaO = Ca^{2+} + 2HCO_3^- + H_2O$$

If the water already contains carbonate alkalinity, the leached calcium will reprecipitate within the concrete matrix as $CaCO_3$ (calcite):

$$Ca^{2+} + 2HCO_3^- = CaCO_3(s) + H_2CO_3$$
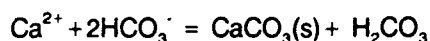
This densification frequently seals the concrete structure in the front zone of deterioration in a self-healing manner. The sealing process, then, migrates toward the outer wall of the concrete structure. Another corrosion process that was of potential concern at Willow Creek Dam was the possibility of corrosion driven by acid generated from the oxidation of $H_2S$. $H_2S$ was accumulating in the deep portions of the reservoir during summer stratification. Heterotrophic bacteria anaerobically reduced organic compounds to form $H_2S$. The oxidation of $H_2S$ can occur through the direct (inorganic) reaction with dissolved oxygen:

$$HS^- + 2O_2 = SO_4^{2-} + H^+$$
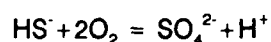
or it can be driven by sulfur oxidizing bacteria:

$$CO_2 + HS^- + O_2 + H_2O = CH_2O \text{ (organic matter)} + SO_4^{2-} + H^+$$
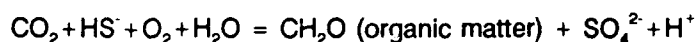
In both cases, the presence of oxygen is required. The process, therefore, was speculated to be limited to the interface of where oxygen and $H_2S$ coexisted, such as the discharge site of the dam seeps and at the locations of the oxycline in the reservoir on the upstream face of the dam. It was also hypothesized that sulfate reduction could also occur within the seepage passages of the dam mass by a resident population of bacteria.

## CHEMISTRY OF SEEPS

The chemistry of the water flowing through the seeps of Willow Creek Dam is variable. Some of the seep waters are similar to the reservoir waters and appear to have a direct path from the reservoir. Other seep waters are highly enriched in dissolved materials. It was speculated that the left embankment waters had a longer residence time within the dam. Much of the compositional differences observed between the reservoir waters and the seep waters could be accounted for by three dominant processes: photosynthesis, decomposition of organic material, and alternation of materials from the dam.

Willow Creek Lake is a nutrient-rich impoundment. The reservoir site was a former feedlot. Little or no effort was made to remove the topsoil from the impounded area before inundation. Additionally, water from the contributing watershed pass through several pasturelands. These lands provide additional nutrient-enriched water to the reservoir, especially during high runoff events. Thermocline development during the spring traps the enriched waters in the hypolimnion into the fall season. Throughout the summer, basic limnological dynamics occur, involving phytoplankton photosynthesis. As the algae die, the organic particulates settle down through the thermocline barrier and eventually decompose within the depths of the hypolimnion. The oxidative decomposition of the phytoplankton debris consumes all of the dissolved oxygen available in the water column. Subsequently, nitrates and sulfates are depleted from the deep portion of the reservoir. The oxidation of organic material using sulfates is particularly important because $H_2S$ gas is a product of the reaction. Also, the carbon dioxide content of the deep reservoir water increases because of the consumption of the organic material throughout the summer. This process results in the lowering of the pH in the hypolimnion.

These changes in water composition formed the basis for two general hypotheses to explain the leaching of the concrete material in the dam. The first hypothesis was that carbon dioxide production resulted in more corrosive waters, and calcium and other elements could be leached from the dam. If the waters became saturated with respect to calcium carbonate as it passed through the dam, precipitation would be expected to occur when the migrating water exited and carbon dioxide degassed when atmospheric conditions were encountered. The second hypothesis was that as $H_2S$ rich waters migrated through the dam, sulfur oxidizing bacteria would use the compound as an energy source to transform dissolved carbon dioxide into cellular material. This process was likely to occur on the surfaces of the dam. In both hypotheses, eutrophication of the reservoir was the underlying case.

Table 1 summarizes the general nature of the chemical changes seen as water moves from the reservoir through the seeps to the downstream portion of the dam. The most striking change in composition is seen in the highly evolved seeps. The initial alkalinity of the reservoir retards the possibility of corrosion within the dam by forcing the reprecipitation of calcite within and on the structure. It was concluded that chemical alterations have been occurring on both the RCC and the grout substrates. Calcite was precipitated in some parts of the system and remobilized from other locations as the seep water and the dam's buffering substrates evolve. The highest chemical alterations, however, were isolated to a small volume fraction of the seep flow (Dymond and Collier, 1987).

Table 1. Example of Chemical Changes Occurring in Willow Creek Dam Seeps.

| PARAMETER | RESERVOIR | LEFT EMBANKMENT | SPILLWAY | RIGHT EMBANKMENT |
|-----------|-----------|-----------------|----------|------------------|
| TDS | 194.0 | 428.0 | 357.0 | 197.0 |
| ALK | 132.0 | 212.0 | 230.0 | 139.0 |
| pH | 7.4 | 9.1 | 8.5 | 7.9 |
| total $CO_2$ | 127.0 | 176.0 | 201.0 | 119.0 |
| Ca | 6.2 | 8.1 | 11.5 | 8.4 |
| $SO_4$ | 3.0 | 55.0 | 22.0 | 22.0 |
| Mg | 15.0 | 16.0 | 19.0 | 14.0 |
| Na | 16.0 | 17.0 | 20.0 | 20.0 |
| K | 3.4 | 12.6 | 5.9 | 5.7 |
| Cl | 2.2 | 14.0 | 1.0 | 5.0 |
| $H_2S$ | 2.2 | 0.4 | 0.3 | 0.4 |

## STUDIES

Preliminary studies of the geochemical and microbiological processes occurring at Willow Creek Dam during 1986 indicated that dissolution of concrete varied from 5 to 82 metric tons per year. Further studies from 1987 through 1989 improved sampling techniques and processing of samples. More detailed analyses of reservoir and seep water better defined the processes affecting the dam materials. The best estimate of the dissolution rate for 1987 and 1988 data was approximately 50 metric tons per year. The most precise 1989 estimate was that 10 to 20 metric tons of total dissolved solids are removed from Willow Creek Dam per year. This 1989 findings also concluded that the concentration of dissolved material has been increasing over mid-1986 measurements. As much as 100 metric tons of calcium may have been redistributed, resulting in the deposition of 250 metric tons of calcite. Additionally, large numbers of acid-producing bacteria are creating acids at the outer faces of the dam (the gallery and the front face) that are exposed to oxygen. The rates of oxidation have not been measured. Most importantly, sulfur oxidation in the interior of the dam could not be attributed to bacterial activities.

The Portland District has taken the findings of the 1986-1989 geochemistry and microbiological studies into account, along with the results of the 11 other geotechnical studies that were performed:

"Based on the 13 completed studies, the major Corps findings are as follows: the precipitation of calcite along the lift joints has resulted in a decrease in seepage rates and this trend is expected to continue; dissolution and redeposition of the dam materials is occurring, but the net effect of the dissolution is not considered a negative process, and in fact, is believed to contribute to the infilling of voids along lift joints with calcite and the subsequent decrease in seepage; the physical analyses of the structure show that current RCC quality is similar to RCC quality following construction in 1984; there is no observed deterioration of the interior mass RCC resulting from dissolution; and, structural stability analyses, using latest test data, show no conditions that do not meet standard Corps of Engineers criteria. The Corps will, however, continue seepage and reservoir water studies to monitor and evaluate any long-term changes." (Portland District, 1990.) "After careful review and evaluation of all studies, reports, and evaluations the Corps of Engineers confirms earlier assessments that the dam is a safe structure and does not present a safety or health hazard. The Corps' assessment that the dam is safe is based upon evaluation of a wide range of investigations that include the Portland District dam safety inspections performed annually since 1984 by teams of trained engineers and scientists; assessment of the structure by higher level organizations with the Corps of Engineers; assessment of the structure by a special Department of the Army investigation team in October 1988; study findings from independent scientists and testing laboratories; and evaluation of the many specific tests performed on the dam itself by Portland District staff." (Portland District, 1990.)

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# DEVELOPMENT OF A STANDARD OPERATING PROCEDURE FOR SEDIMENT EVALUATION OF HABITAT AND REHABILITATION PROJECTS WITHIN THE ST. PAUL DISTRICT, U.S. ARMY CORPS OF ENGINEERS

by

Dennis Anderson[1] and Robert Whiting[2]

## INTRODUCTION

The Water Resources Development Act of 1986 authorized the Upper Mississippi River Environmental Management Program (UMRS-EMP). This $260 million, 10-year program includes "a program for the planning, construction, and evaluation of measures of fish and wildlife habitat rehabilitation and enhancement." The UMRS-EMP is funded through Construction General. There are presently, within the St. Paul District (CENCS), 17 projects at various stages of planning, design, and construction, at an estimated construction cost of $19,573,000. These habitat rehabilitation and enhancement projects (HREP's) have been conceived by the individual states and by the U.S. Fish and Wildlife Service. Many of these proposed HREP's are located in backwaters of the Upper Mississippi River where there are silt and clay substrates with low to moderate levels of contaminants.

The program is characterized by tight time schedules, limited planning funds, and multiple projects being planned concurrently by different Federal and state agencies. A uniform, expedient, and economical approach to the evaluation of sediment contaminants was necessary. CENCS developed a Standard Operating Procedure (SOP) for sediment evaluation in this program for use within the St. Paul District.

## APPROACH

### Tiered Evaluation

The SOP consists of a tiered testing approach with a decision-making process occurring at the end of each tier. This approach, which is generally consistent with national guidance concerning dredged material evaluations, uses tests of increasing complexity and sophistication to reach decisions with greater degrees of confidence. This approach provides a defensible and technically sound rationale for regulatory decision-making and allows for economically efficient decisions as early as possible in the planning process. More effort, funding, and sophisticated tests are concentrated on projects of greater concern.

The recommended components for the three tiers of evaluation are summarized in Figure 1. Tier I is an initial evaluation using only existing information, including the following: (1) particle size gradation, which can indicate a potential for contaminant levels; (2) available sediment quality data from within or near the project area; (3) historical input information, including type and proximity to point and non-point discharges, spills, and other sources of pollution; (4) sedimentation history to determine when and how the material to be dredged has accumulated; (5) description of project area, including identification of biologically sensitive areas; and (6) project description, including quantity of dredged or fill material, dredging and disposal methods, and sites being considered. Tier II comprises the standard bulk chemical analysis of sediments and a predictive calculation of neutral organics bioaccumulation

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| Tier | Basis For Evaluation | Assessment | Implementation Strategy |

**Tier I** — Initial Evaluation
- Existing sediment quality data
- Particle size of material
- Historical input
- Siting
- Project alternatives

Sufficient information to evaluate project? — no → Proceed to next tier of testing
yes ↓

Reason to believe sediments contaminated? — no → Proceed with project, without special restrictions
yes ↓

Can restrictions or modifications be made to alleviate concern? — no → Abandon project
yes → Proceed with project, with special restrictions or modifications

**Tier II** — Bulk chemistry; Predicative calculation of neutral organics

Concern with levels of contaminants? — no → Proceed with project, without special restrictions
yes ↓

Sufficient information to evaluate project impacts? — no → Proceed to next tier of testing
yes ↓

Can restrictions or modifications be made to alleviate concern? — no → Abandon project
yes → Proceed with project, with special restrictions or modifications

**Tier III** — Modified elutriate/mixing zone; Column settleability; Chronic toxicity tests:
- 10-day Daphnia magna partial life cycle test
- 14-day Chironomus tentans partial life cycle test
- Laboratory determination of bioaccumulation potential
- 10-day Pimephales promelas exposures.

Concern with toxicity or bioaccumulation? — no → Proceed with project, without special restrictions
yes ↓

Can restrictions or modifications be made to alleviate concern? — no → Abandon project
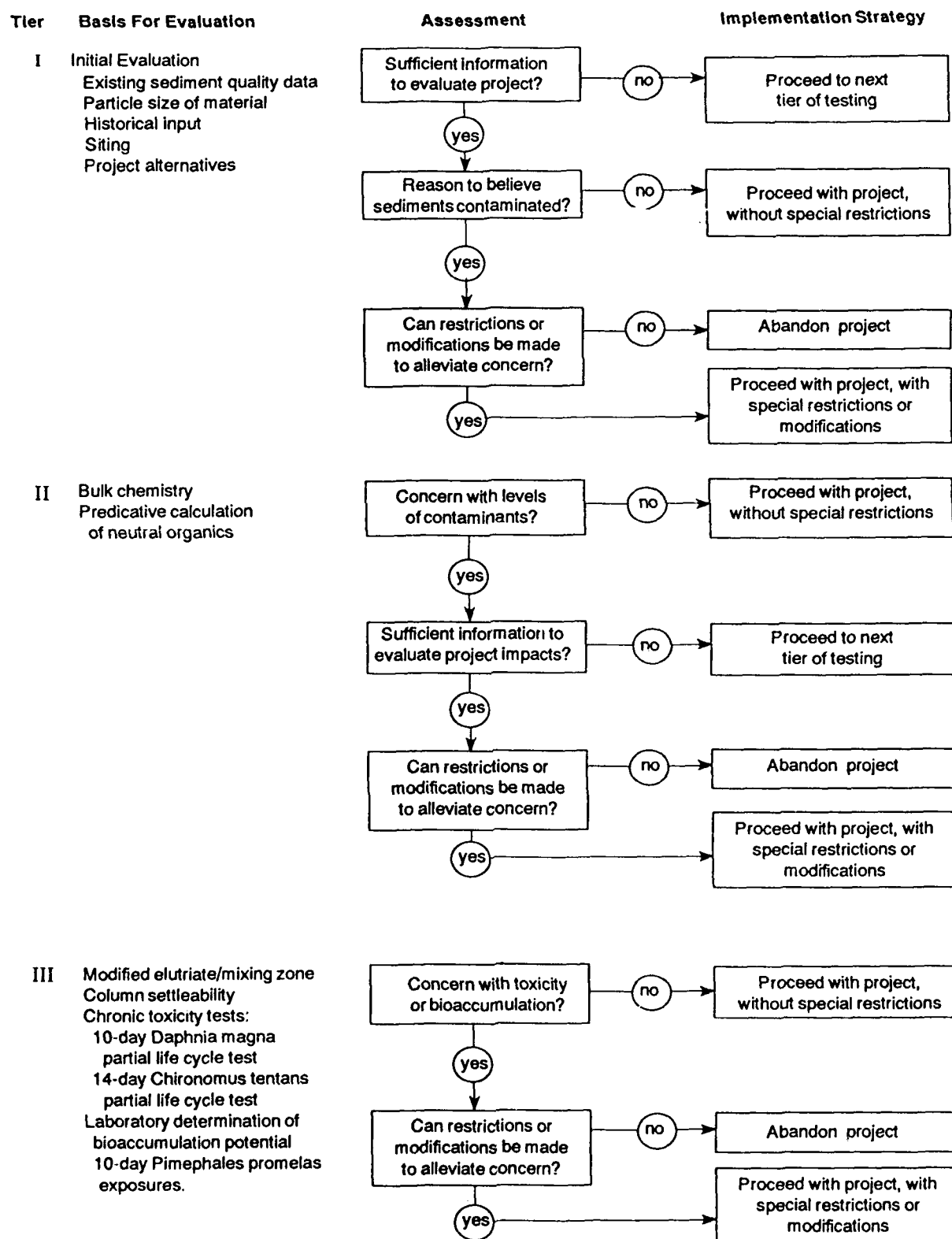yes → Proceed with project, with special restrictions or modifications

Figure 1.  Tiered Sediment Testing Protocol for HREP's

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potential. Tier III involves more sophisticated tests, including the modified elutriate, column settleability,and biological response tests. The biological response tests concentrate on chronic toxicity and bioaccumulation potential.

Acute toxicity testing was not recommended for HREP, because previous data has shown that it is extremely rare to find backwater sediments, in the CENCS portion of the UMR, that produce acute toxicity. In addition, some information is obtained on the acute toxicity of sediments from the chronic exposures. Some of the high levels of ammonia nitrogen in backwater sediments that have been recorded may be an exception, capable of causing acute toxicity. In order to address this concern, the standard ammonia bulk chemistry procedure was modified to an elutriate procedure, to allow a direct comparison to water quality criteria and an early determination whether ammonia could pose a problem. A benthic community analysis of the project area may be desirable for certain projects and could be conducted as part of the tier III evaluation to assist in the interpretation of the chronic toxicity data.

There are limitations with the recommended protocol. This protocol only deals with evaluating the potential impacts of contaminants on aquatic biota from open water disposal, or effluents from a containment area or from re-exposing contaminants at a disposal site or at a dredge cut area. A variety of other factors, including physical impacts, has to be considered when evaluating a project. In addition, there may be other contaminant concerns associated with a particular project.

## Decision-Making

From the tier I information, a determination is made about the potential for contaminants to be present at levels of concern. If there are concerns, the specific contaminants and types of problems associated with each of the project alternatives are identified. In making this determination, the adequacy of the data has to be considered. A lack of adequate information would constitute a reason to be concerned about contaminants. If there is no concern with contaminants, then proceed with the project planning, without special project restrictions.

Selection of one of the project alternatives and the project design is made based on other factors. If there is a concern with contaminants, then the next step is to determine whether there is sufficient information to evaluate the potential effects of the project. If the answer is no, then proceed to the next tier of testing. If the answer is yes, then determine whether restrictions that are economical and feasible from an engineering standpoint can be made to the project to alleviate the contaminants concerned. If the answer is no, then the project or that portion of the project of concern is abandoned. If the answer is yes, then proceed with the project planning, including the appropriate restrictions.

This type of decision-making is followed after each tier, except that tier III would not have a determination on whether there was adequate information. The reason for this exception is that there has to be some end point in the testing, at which point a final decision on the project has to be reached.

The tiered approach was not intended to be rigid. In all cases, the tier I initial evaluation is performed. Beyond tier I, decisions to continue with further testing, and the specific tests to be performed, are made on a project specific basis. In most cases, the tiers are followed in sequence, with an interagency decision process, as outlined in the next paragraph, occurring at the end of each tier. However, the system has to be flexible, and there may be instances when the initial evaluation in tier I indicates that it is advisable to skip tier II testing and go directly to tier III or perform a combination of tier II and tier III tests. In addition, not all the components within a given tier, especially tier III, are performed for all projects in which a decision is made to advance to that tier. This has to be decided for each particular project based on the results of the earlier tiers and other factors. For example, even if a decision is reached to prcceed to tier III testing, if the results from the bulk chemistry and predictive calculations of bioaccumulation potential from tier II do not indicate a concern with bioaccumulation potential, the laboratory determination of bioaccumulation potential in tier III is not performed. Only those tests in a tier that are necessary to make a technically sound determination are conducted.

Interagency coordination is an integral part of the decision-making process. When the results of a tier are obtained, the Corps of Engineers evaluates the results and makes a preliminary determination. The results and the preliminary determination are then coordinated with all the agencies having regulatory authority, and a mutually agreed upon decision is made. The agencies included are the U.S. Environmental Protection Agency, the U.S. Fish and Wildlife Service, and the appropriate State agency having regulatory authority for the particular project.

## SEDIMENT SAMPLING PROTOCOL

<u>Sampling Design and Numbers</u>

The nature of the program and the tight funding and schedules are considered in the sampling design. In designing the sampling protocol for a particular project, two major factors are considered; specifically, the anticipated analytical variability and the spatial heterogeneity. Measures to address analytical variability are included in the quality assurance/control section of this report. To handle horizontal heterogeneity, the most frequently used approach is stratified random sampling. This is done to optimize sampling to the geographic areas of greatest concern. The reasons for stratifying the sampling can include proximity to a potential source of pollution, different sediment textures within the dredge cut(s), existing data indicating potential hot spots, different sedimentation history within the dredge cut(s), or any other reasons that would cause one to suspect and be able to predict spatial heterogeneity.

If there is no basis for stratifying the sampling, then a completely randomized sampling is most appropriate. The number of sampling sites should be representative and should be decided on a project specific basis considering the degree of areal heterogeneity anticipated, the degree of contamination expected, and the quantities of dredged material and the disposal methods being proposed. At a minimum, three cores are collected from the project area.

The other source of spatial variability that is considered in designing the sampling effort is vertical heterogeneity. A major concern expressed by the various agencies has been the potential to re-expose sediments with higher concentrations of contaminants that are presently sequestered within the proposed dredge cut area. This concern is based on the fact that higher levels of persistent chemicals, such as polychlorinated biphenyls (PCB's), were recorded in fish and surficial sediments in the 1960's and 1970's.

Stratifying the sampling with depth quickly multiplies the amount of sampling effort and subsequently the cost. Care is exercised when deciding if vertical stratification is necessary. In evaluating whether there is potential for and concern with vertical heterogeneity, the sedimentation history and sediment stratigraphy for the area have to be evaluated. Where there is a potential for the dredge cut to expose a more contaminated layer, at a minimum two composite samples are taken, one from around 1 to 2 feet above and below the dredging depth and the other from the remaining core. If additional stratification is warranted, additional sub-sampling based on a visual examination is done.

<u>Sample Collection and Storage Methods</u>

Sediment samples for analytical work are collected with wide mouth corers (2 inches or greater). Samples for organic analysis are collected with a stainless steel corer, and samples for metal analysis are collected with a polyvinyl choloride (PVC) or similarly inert corer.

Sediment samples are collected and stored at 4°C in glass containers with teflon-lined caps for analysis of organics. Storage containers for analysis of metals are either linear polyethylene containers or glass containers with teflon-lined caps. The elutriate procedure is initiated within one week of collection. Water samples resulting from the elutriate procedure are stored and preserved as specified for normal water samples in EPA (1983) and Plumb (1981).

Samples for biological response testing are collected and stored in linear polyethylene containers or stainless steel containers. The containers are filled to the top, leaving no air space. The samples are maintained on ice and delivered to the laboratory within 24 hours of collection. At the laboratory, the samples are homogenized with a commercial mixer equipped with stainless steel bowl and paddles and stored at 4°C. All tests are initiated as soon as practical after collection, but no later than seven days after collection.

## ANALYTICAL PROCEDURES

### Bulk Chemical and Physical Characterization

A standard list of chemical and physical characteristics is run on all samples collected. Additional parameters are added to evaluate a specific project if it is suspected that other contaminants may be present at levels of concern. The analytical methods as described by Plumb (1981), EPA (1983), EPA (1982), and EPA (1986) are followed.

### Modified Elutriate

The modified elutriate procedure as described in Environmental Effects of Dredging Technical Notes - EEDP-04-2 (WES 1985) is followed. The supernatant samples are then treated and analyzed following the methods for water samples (EPA 1983). For analysis of the dissolved constituents, the samples are first filtered (0.45 um filters) and/or centrifuged, depending on the specific parameters to be tested. Samples for analysis of total concentrations would undergo appropriate digestion (EPA 1983) prior to analysis. The parameters tested in the filtered or whole supernatant are those found to be of potential concern from the existing data base, or based on the results of the bulk chemistry obtained in the tier II testing.

### Column Settling Test

The column settling test is designed to provide a way to predict the concentration of suspended solids in an effluent and to define the settled behavior of a particular sediment. The protocol is described in Environmental Effects of Dredging Technical Notes EEDP-04-2 and EEDP-04-3 (WES 1985).

### Predictive Calculation of Neutral Organics Bioaccumulation Potential

Neutral organics chemicals, such as PCB's, are distributed within an aquatic ecosystem, primarily in the lipids of organisms and in the organic carbon fraction of the sediment. The predictive model of McFarland (1987) is used to calculate the maximum possible concentration that could result in an organism's lipid and, subsequently, the whole-body bioaccumulation potential.

$$WBP = 1.72*FL*(C_s/FOC)$$

$WBP$ = Maximum whole-body bioaccumulation potential (wet weight - in the same units of concentration as $C_s$).

$FL$ = Decimal fraction of an organism's lipid content (wet weight).

$C_s$ = Concentration of chemical in the sediment (dry weight - any unit of measurement).

$FOC$ = Decimal fraction of organic carbon content of the sediment (dry weight).

## QUALITY ASSURANCE/CONTROL PROTOCOL

Potential contractors that would do any of the analytical work are required to have a comprehensive quality assurance/control program, including documentation following the procedures of EPA (1979). A laboratory audit of a potential contractor's laboratory is performed. The potential

contractor is required to correct any deficiencies noted during the audit prior to any sample analysis. In addition to evaluating a potential contractor's quality assurance program, the quality assurance measures including duplicate or split samples, laboratory blanks, replicate analyses, and blind samples are run routinely with every batch of samples analyzed by the contractor's laboratory.

## BIOLOGICAL RESPONSE TESTING

All the biological response testing described below involves the use of whole sediments (solid and suspended phases). The use of whole sediments is a good approach when evaluating the effects of open water disposal because it simultaneously looks at the water column impacts and the benthic impacts at the disposal site. However, it is a very conservative approach when evaluating the effects of effluents or long-term runoff from containment areas, where the major concern may only be water column effects. Standard reference toxicant bioassays, with 1-octanol, are also conducted to assess the sensitivity of the laboratory test organisms (Busacker and Anderson 1989). A control, a reference, and the test sediment exposure are run for all the biological response testing.

### Chronic Toxicity Testing Protocol

Two chronic toxicity tests are used to determine the chronic toxic effects of chemicals sorbed to the sediments. The 10-day *Daphnia magna* partial life cycle test as described in Busacker and Anderson (1989) and adapted from the work of Nebeker et al. (1984) is used. In addition, the 14-day *Chironomus tentans* partial life cycle test as described in Busacker and Anderson (1989) and adapted from the work of Mosher, Kimerly and Adams (1982)is used.

### Laboratory Determination of Bioaccumulation Potential

The 10-day *Pimephales promelas* procedure adapted from Mac et al. (1984) and described in detail in Busacker and Anderson (1989) is used. An alternative species is selected if polyaromatic hydrocarbon (PAH) compounds are the major concern, because of the ability of fathead minnows to metabolize PAH compounds. If bioaccumulation is a major concern, then the procedure is expanded to provide kinetic uptake information. A minimum of 5 time interval exposures is done (i.e., 2, 4, 6, 8, and 10 days). Calculation of potential maximum bioaccumulation is then accomplished using a linear regression technique and an iterative process to estimate the steady state value (Busacker and Anderson 1989).

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# DROUGHT MANAGEMENT PLANS FOR
## RESERVOIR PROJECTS IN THE BALTIMORE DISTRICT

by

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## Introduction

The Corps of Engineers has the authority to deviate from approved regulation plans for reservoir projects to provide assistance during droughts. Under direction of ER 1110-2-1941, drought management plans are prepared for each reservoir project.

The Baltimore District's civil works boundaries include the Susquehanna and Potomac River basins. There are 12 reservoir projects in the District containing controlled storage which could provide drought assistance. The District was divided into six subbasins for the purpose of preparing drought management plans. During a drought, the reservoirs in each subbasin are regulated in a system concept to achieve maximum benefits.

This paper discusses the content of the drought management plans, coordination of the plans with other agencies, and effects of implementation of drought management plans during the drought of 1988.

In general, the drought management plans contain criteria for implementation of the plan, a modified reservoir regulation plan for drought management, and coordination of drought management activities with other agencies.

## Triggering Criteria

Several drought criteria were considered before choosing one to be the trigger for implementation of the drought plan. Among those considered were stream flow, ground water level, soil moisture, Palmer Drought Severity Index, and precipitation deficit. It was desired to find an indicator which would provide an early detection mechanism so that the drought plans could be properly implemented, while not being so sensitive that it would trigger implementation of the plans unnecessarily. In addition, the indicator should predict the onset of a regional drought and not a localized event.

A study was performed for the Pennsylvania Department of Environmental Resources by Penn State University which investigated the sensitivity, reliability and consistency of regional drought indicators in Pennsylvania. The historical record of the aforementioned drought indicators was analyzed to determine which of them would give the best regional indication of the onset of a drought. This study concluded all of these indicators provide useful information about the onset and termination of a drought. Because of the availability of data, precipitation deficit was chosen as the trigger for implementation of the drought management plans. Precipitation deficits also gave the earliest indication of the onset of a drought.

The Penn State study also set triggering criteria for various drought conditions. The drought watch, drought warning and drought emergency levels were set at the 75, 90, and 95 percent quantiles, respectively. These levels were established for durations of 3 to 12 months and are shown in Table 1. The study suggested using 3-month duration to get an early indication of an onset of a drought to begin monitoring the situation. The study suggested using the values for 6-month duration to begin any

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## TABLE 1

### Drought Assistance Triggering Criteria†

Ratio of Precipitation Deficit to Normally Expected Precipitation (Percent)

| Duration (months) | Drought Watch | Drought* Warning | Drought* Emergency |
|---|---|---|---|
| 3 | 18.0 | 31 | 35.0 |
| 4 | 15.5 | 27 | 32.0 |
| 5 | 14.0 | 24 | 29.0 |
| 6 | 12.5 | 22 | 26.5 |
| 7 | 12.0 | 21 | 25.0 |
| ? | 11.5 | 20 | 24.0 |
| 9 | 11.0 | 19 | 23.0 |
| 10 | 10.5 | 18.5 | 22.0 |
| 11 | 10.5 | 18 | 21.0 |
| 12 | 10.0 | 17.5 | 20.5 |

*Attaining these threshold values dces not automatically constitute an emergency. There must be a formal declaration. These values are for informational purposes.

†These criteria were developed by the Pennsylvania State University for PennDER. These criteria are used as drought assistance triggers because they displayed uniform regional characteristics, while other criteria (groundwater levels, streamflow, etc.,) varied according to geographic location.

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actions. We have followed the recommendations of the Penn State study in choosing triggering criteria for implementation of our drought plans.

## Water Control Plan

When drought triggering criteria are met, the regulation of reservoir changes from following the normal regulation plan to following a modified plan to produce maximum drought-related benefits. Reservoirs in the Baltimore District will offer drought-related assistance in two steps, the first step when "drought watch" conditions are experienced and second when a "drought emergency" has been formally declared.

When the "drought watch" threshold, which translates to a precipitation deficit of 12.5% below normal for the previous 6 months, is met, Phase I of our drought management plans is implemented. Phase I includes the following actions:

a. At all projects having seasonal pools, raise pool to the summer pool level, if they are not already there.

b. At those projects where we have identified that we can store surplus water without infringing on other project purposes, raise the pool to that specified level. (See Table 2 for list of projects.) This is always dependent on the availability of inflow.

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# Drought Management Plans

## TABLE 2

## DROUGHT CONTINGENCY STORAGE

| PROJECT | NORMAL | | DROUGHT CONTINGENCY | |
|---|---|---|---|---|
| | POOL ELEV (FT NGVD) | STORAGE (AC FT) | POOL ELEV (FT NGVD) | STORAGE (AC FT) |
| EAST SIDNEY LAKE | 1150.00 | 3350 | 1151.00 | 3550 |
| WHITNEY POINT LAKE | 973.00 | 12500 | 973.50 | 13100 |
| ALMOND LAKE | 1260.00 | 1105 | 1261.00 | 1261 |
| STILLWATER LAKE | 1572.00 | 343 | 1580.00 | 1250 |
| SAYERS DAM | 630.00 | 28800 | 631.00 | 30550 |
| RAYSTOWN LAKE | 786.00 | 514000 | 786.50 | 518250 |
| INDIAN ROCK DAM | 372.00 | 0 | 390.00 | 1010 |
| TIOGA LAKE | 1081.00 | 9500 | 1081.50 | 9750 |
| HAMMOND LAKE | 1086.00 | 8850 | 1086.50 | 9150 |
| COWANESQUE LAKE | 1080.00 | 31335 | 1081.00 | 33600 |
| TOTAL | | 609783 | | 621471 |

c. Call meeting of Baltimore District Drought Committee to discuss drought situation, actions taken, and anticipated future actions. This committee is made up of personnel from Water Control Management Section, Planning Division, Emergency Management Branch, Public Affairs Office, Operations Division, and Office of Counsel. The committee will then brief the District Engineer on the situation.

d. North Atlantic Division will be advised of the situation and all actions taken. Situation reports will be prepared by Emergency Management Branch, if necessary.

e. Members of the District Engineer's Drought Advisory Committee will be advised of actions, and will be asked if they have taken any action. This committee includes representatives of state resource management agencies, river basin commissions, National Weather Service, and Geological Survey. If necessary, a formal meeting of this committee will be called.

As the drought continues, releases from reservoirs will be maintained at a predetermined minimum rate, or higher. This will continue until the pool falls to the summer pool elevation. At that time, the operation will go to an inflow equals outflow mode. Those projects with a flow augmentation purpose will continue to release the minimum required flow to meet downstream water quality requirements.

The next phase of drought assistance will not begin until a drought emergency has been declared by a governor, river basin commission etc. After a formal declaration has been made, drought contingency releases may be requested if it can be shown that all other sources of water have been exhausted. Requests will be made in the following sequence:

(1) Consumer contacts river basin commission, state or local agency requesting release from Corps project.

(2) Agency will contact Baltimore District Engineer.

(3) District Engineer will convene a meeting of an advisory committee to discuss this request. Committee could include representatives of river basin commissions and state resource management agencies. Following the meeting, the District Engineer will decide if the release will be made.

(4) If the release will be made, a contract between the agency making the request and the Corps must be drawn up and signed before the releases may begin. The Corps must be reimbursed for the cost of the water. This reimbursement must be at least equal to a proper share of annual, joint-use O&M cost and major replacement expense plus revenues foregone and any other costs directly attributable to making releases.

(5) The Corps will then decide from what reservoir(s) the release will be made.

This assistance will rarely be called for since most water supplies are from ground water sources and those using surface water have intakes at very low elevations within the streams.

## Effectiveness of Plans During Drought of 1988

The spring and summer of 1988 was the driest period in the Baltimore District since the drought of 1965-66. This followed a winter which produced much below average snow fall. By 1 April, "drought watch" conditions existed throughout the Baltimore District. At that time, the drought management plans for all reservoir projects were implemented. At that time, all projects were raised to summer pool, and the "drought contingency" water was stored at those projects which were identified as able to store this water. This added an additional 8350 acre feet of storage in the District reservoirs.

The drought was most severe in the West Branch Susquehanna, Juniata and North Branch Potomac Basins. However, throughout the drought the Baltimore District was able to maintain its established minimum flows at most of our projects without affecting recreation. This was because of the drought contingency water which was stored at the projects. The only problem area was the North Branch Potomac basin. At Jennings Randolph Lake, the boat launch area was closed in early August, about one month earlier than normal, because of the low pool elevation. At Savage River Dam, 4,600 acre-feet, about 25 percent of the total storage in the project, were released in June for an international white water event. These two projects' main purpose is flow augmentation for water quality improvement in the Potomac basin.

The combination of the drought conditions, and the loss of storage due to the white water competition caused the depletion of almost all water quality storage available in the two projects. Because the drought was not severe in the Washington, D.C. area, the D.C. area water supply storage owners gave the Corps permission to utilize a portion of their water supply storage in Jennings Randolph Lake for water quality purposes. This allowed us to continue our water quality operations on the Potomac River throughout the remainder of the low flow period.

Overall, the drought plans worked very well during the Drought of 1988. The biggest benefit was from water quality because of the ability to maintain minimum flows from the projects. In the headwater areas of the Baltimore District, releases from its reservoirs provided up to 75 percent of the flow in the river, and at other locations 25-30 percent of the flow was due to the augmentation provided by reservoirs in the Baltimore District.

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# ASSESSMENT OF SEDIMENT-RELATED ENVIRONMENTAL PROBLEMS IN A CENTRAL WISCONSIN RESERVOIR

by

Douglas Gunnison and John W. Barko[1]

## BACKGROUND

In 1937 the Wisconsin Valley Improvement Company (WVIC) constructed the 2,760-ha Big Eau Pleine Reservoir by impounding the Big Eau Pleine River. The reservoir was created to assist in providing uniform flows in the Wisconsin River, and has had a subsequent history of major winter fish kills and summer algal blooms (Shaw and Powers Undated). Based on results of previous studies, the winter fish kill problem is believed to result from an oxygen sag which develops periodically under the ice in the upper reaches of the reservoir, and then moves downstream as water is withdrawn from the project (Gunnison and Barko 1988).

The U.S. Army Engineer Waterways Experiment Station (WES) was asked to examine results of previous studies and other available data to evaluate earlier recommendations for water quality improvement (Gunnison and Barko 1988). Results of this effort indicated that the reservoir is a sink for total phosphorous, but releases organic matter in the form of biochemical oxygen demand. Insufficient information was available to permit a determination of the environmental factors contributing to dissolved oxygen sag development and propagation. However, additional information related to the dissolved oxygen depletion problem is being obtained from intensive field investigations initiated in 1989. The objective of this initial report is to assess the role of sediment dynamics in dissolved oxygen depletion during the winter in the Big Eau Pleine Reservoir.

## METHODS AND MATERIALS

During May 1989, sediment samples were taken at 37 locations along the length of the reservoir. Multiple samples were taken along transects at river miles 0.3, 8.0, 12.0, 16.0, and 17.4. The position of each sampling location and depth was accurately determined for later resampling activities. Two types of samples were taken at each location. These included 5-cm core samples taken to the depth of compacted sand, or to a maximum of about 25 cm (core length).

Core samples were measured for length, and then sectioned vertically into 0-5 and then 10 cm strata for determinations of physical characteristics. Analyses included bulk density and organic matter content; these data were used to characterize patterns of sediment transport and focussing in the reservoir.

To determine the nature of the sediments in intimate contact with the overlying water, several chemical characteristics of the surficial sediments were examined. Composited surficial sediment samples were analyzed for chemical variables using suitable precautions to maintain the anaerobic integrity of the sediments. With interstitial water fractions, Fe, Mn, soluble reactive phosphorus (SRP), ammonium nitrogen (NH4-N), chemical oxygen demand (COD), and dissolved organic carbon (DOC) concentrations were determined along with conductivity and pH.

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## RESULTS

### Sediment Density and Organic Matter Content

Table 1 presents sediment density information obtained from core samples taken at various points along the length of the reservoir. There were four locations where sediments had similar low densities at each of the three depths sampled. These were at river mile 3.0, river mile 10.0, river mile 12.0, and river mile 14.0. Average densities for the entire 0-15 cm depth at these locations were 0.42 $\pm$ 0.04 g/ml at river mile 3.0, 0.33 $\pm$ 0.02 g/ml at river mile 10.0, 0.64 $\pm$ 0.04 at g/ml river mile 12.0, and 0.54 $\pm$ 0.06 g/ml at river mile 14.0. While there were sporadic incidences of values similar to these at other locations, none of the other areas had deposits of material having low density material homogeneously distributed over the entire 15-cm depth.

Table 2 gives a comparison of sediment organic matter content in core samples from the same locations along the reservoir. There were several locations where organic matter content did not vary over the 0-15 cm depth. These were at river miles 3.0, 4.0, 9.0, 10.0, 12.0, 12.5, 16.0, and 17.4. Maximum values of sediment organic matter content occurred at river miles 8.0 and 10.0. Minimum sediment organic matter content occurred at river mile 13.7, but was restricted to the upper 0-5 cm. Consistently low sediment organic matter content occurred over the entire 0-15 cm depth at river miles 3.0 and 17.4.

### Sediment Chemistry

Figure 1 depicts $NH_4$-N, SRP and DOC levels in sediment interstitial water along the course of the Big Eau Pleine River from 0.3 miles above the dam to mile 17.5, where the uppermost sample was taken. The pattern encountered for $NH_4$-N and SRP was quite erratic. With the exception of a peak value of 31.7 mg/l achieved at mile 2, $NH_4$-N values were less than 20 mg/l over the course of the reservoir length. SRP peaks of 1.2 mg/l and 0.88 mg/l occurred at mile 8.0 and 12.0, respectively. A peak DOC value of 87.2 ml/l, more than twice the magnitude of concentrations over the first 8 miles, occurred at mile 10 in a broad-shouldered peak extending between miles 9 and 14.

Iron, manganese, conductivity, and COD in interstitial waters also demonstrated distinct peaks at the 10.0 mile mark (Figure 2). This was particularly pronounced for interstitial water iron, with a peak value nearly three times the next closest value. Lower-level peaks for each of these constituents occurred between miles 2 and 7 above the dam.

## DISCUSSION

The accumulation of reduced iron and manganese under anaerobic conditions in the interstitial waters of flooded soils and sediments is a well-known phenomenon (Ponnamperuma 1972; Yoshida 1975; Brannon, Chen, and Gunnison 1985), and it is assumed that all of the iron and manganese measured in the interstitial waters obtained in this study were in reduced form. The occurrence of a peak value for conductivity in the same areas as peaks for iron and manganese reflects high concentrations of ionized substances in the water. High levels of DOC and COD were found in sediment interstitial water in areas of the reservoir that may receive high inputs of organic matter (Gunnison and Barko 1988). The occurrence of peak values of DOC in combination with reduced metals in the mile 10.0 to 12.0 area is important in view of the low density and, thus, potential erodability of sediment in this area.

Based on the data obtained, we made the following calculation for the oxygen demand released by potential resuspension of the upper 15 cm of sediment in the mile 10 reach of the reservoir (this depth is justified by the homogeneity and low density of the sediment in this area).

1. The volume of potentially suspended sediment per $m^2$ surface area is 150 liters (100 cm x 100 cm x 15 cm = 150,000 $cm^3$/1000 $cm^3$/l = 150).

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Table 1

Comparison of Density in Upper 15 cm of Sediment Cores Taken Along the Course of the
Big Eau Pleine Reservoir.*

| Sample Location (River Mile) | Density, (g/ml) at Depth Interval | | |
|---|---|---|---|
| | 0 - 5 cm | 6 - 10 cm | 11 - 15 cm |
| 0.3 | 0.94 ± 0.26 | 1.15 ± 0.12 | 1.20 ± 0.26 |
| 3.0 | 0.38 | 0.38 | 0.49 |
| 4.0 | 0.81 | 0.91 | 0.87 |
| 8.0 | 0.74 ± 0.23 | 0.58 ± 0.02 | 0.74 ± 0.11 |
| 9.0 | 1.00 | 1.28 | 1.23 |
| 10.0 | 0.32 | 0.30 | 0.38 |
| 11.0 | 0.67 | 1.13 | 1.31 |
| 12.0 | 0.56 ± 0.15 | 0.68 ± 0.18 | 0.67 ± 0.12 |
| 12.5 | 1.11 | 1.05 | 1.12 |
| 13.7 | 0.93 | 0.85 | 1.28 |
| 14.0 | 0.44 | 0.52 | 0.66 |
| 16.0 | 1.37 ± 0.18 | 1.07 ± 0.12 | 1.01 ± 0.10 |
| 17.4 | 1.06 ± 0.06 | 1.24 ± 0.14 | 0.94 ± 0.07 |

*Locations where samples were taken across a transect are presented as means ± standard errors. The remaining samples are data obtained from single analyses.

## Table 2

### Comparison of Organic Matter in Upper 15 cm of Sediment Cores Taken Along the Course of the Big Eau Pleine Reservoir.*

| Sample Location (River Mile) | Organic Matter (Percent) at Depth Interval | | |
|---|---|---|---|
| | 0 - 5 cm | 6 - 10 cm | 11 - 15 cm |
| 0.3 | 5.45 ± 0.86 | 3.09 ± 1.20 | 2.83 ± 1.36 |
| 3.0 | 1.45 | 2.37 | 2.07 |
| 4.0 | 3.88 | 3.55 | 4.07 |
| 8.0 | 9.49 ± 0.48 | 10.20 ± 0.60 | 7.92 ± 1.23 |
| 9.0 | 3.05 | 2.02 | 2.87 |
| 10.0 | 8.33 | 9.65 | 8.59 |
| 11.0 | 5.48 | 4.04 | 2.55 |
| 12.0 | 5.82 ± 0.90 | 5.72 ± 0.93 | 6.17 ± 0.25 |
| 12.5 | 4.51 | 3.22 | 3.61 |
| 13.7 | 0.32 | 5.41 | 5.79 |
| 14.0 | 2.67 | 6.86 | 6.58 |
| 16.0 | 2.38 ± 0.18 | 2.69 ± 0.66 | 3.14 ± 0.67 |
| 17.4 | 2.14 ± 0.46 | 2.34 ± 0.99 | 3.04 ± 2.03 |

*Locations where samples were taken across a transect are presented as means ± standard errors. The remaining samples are data obtained from single analyses.

Figure 1. Nutrient Levels in Interstitial Waters of Sediment Samples Taken Along the Length of the Big Eau Pleine Reservoir: (a) Ammonium-nitrogen ($NH_4$-N); (b) Soluble Reactive Phosphorus (SRP); and (c) Dissolved Organic Carbon (DOC). Bars Represent Variation Obtained When Samples Were Taken Across Transects.
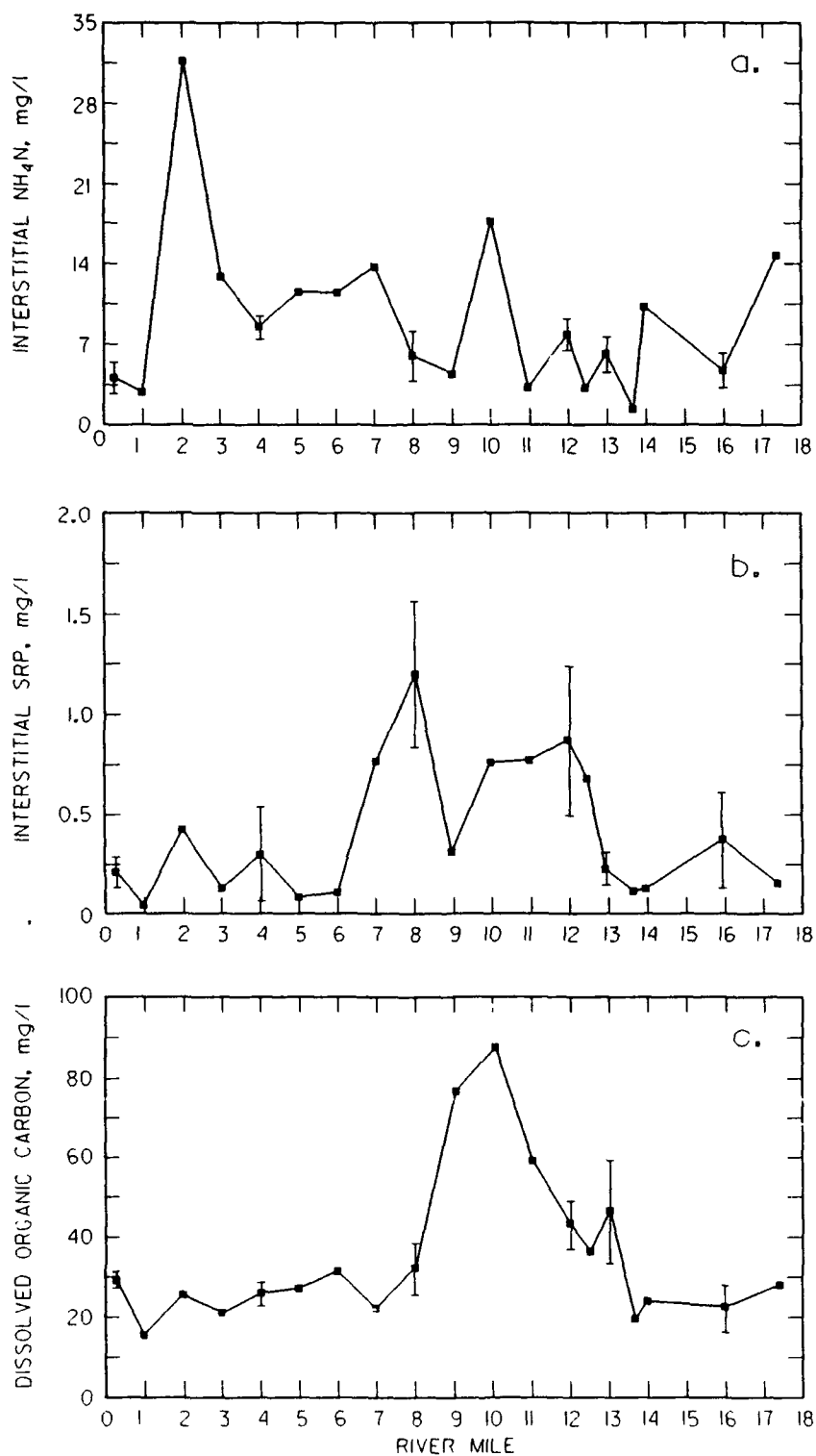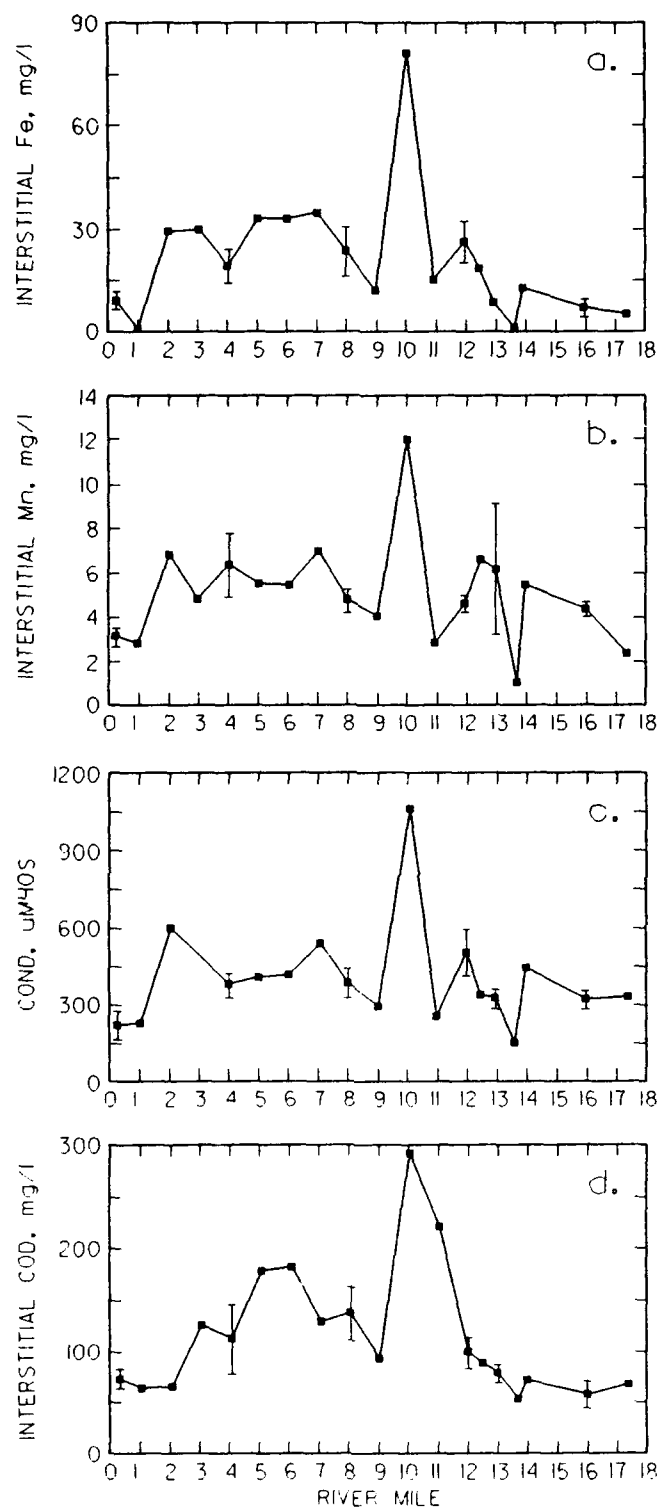
Figure 2. Metal, Conductivity, and Chemical Oxygen Demand (COD) Levels in Interstitial Waters of Sediment Samples Taken Along the Length of the Big Eau Pleine Reservoir: (a) Iron; (b) Manganese; (c) Conductivity; and (d) COD. Bars Represent Variation Obtained When Samples Were Taken Across Transects.

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2. Interstitial water content of 150 liters of sediment at a density of 0.32 g/ml is 102 liters (68% of sediment volume).

3. The COD of the sediment interstitial water at mile 10 is 292 mg $O_2$ per liter. Thus, 292 mg $O_2$/l x 102 liters = a total COD of 29,784 mg $O_2$.

4. At 4 °C, a liter of water holds 13.1 mg $O_2$ at saturation. Thus, resuspension of sediment to a depth of 15 cm could completely deoxygenate 2,274 liters of $O_2$-saturated water.

5. The water depth during winter drawdown at mile 10 is approximately 4 m (per WVIC), but all of the water is in the main channel at this point. This means that resuspension of a single layer of sediment 1 $m^2$ in area by 15 cm deep at mile 10 could remove all of the oxygen from more than 1/2 of the overlying water column. It is important to note that most of the sediment suspended in the main channel could possibly be contributed by material removed from the shallower sides of the main channel, in addition to sediment in the main channel itself.

The findings of this study lend support to the hypothesized explanation of oxygen sag formation in the upper reaches of the Big Eau Pleine River (Gunnison and Barko 1988). Hypothetically, organic matter originating from dairy cattle farms in the upstream area enters the Big Eau Pleine River with spring runoff, and this material, possibly along with algal cells, is deposited as a low-density sediment in the upper reaches of the project. Microbial anaerobic processes, fueled by this organic matter, impart high levels of oxygen-demanding, reduced inorganic species (primarily ferrous iron) and easily-degraded organic substances (primarily volatile fatty acids and alcohols) within the sediment interstitial waters.

This is reflected in the high levels of iron, manganese, and COD in the mile 10 to 12 area of the Big Eau Pleine River. These materials remain until some process dislodges the sediment. One possible source of disturbance is the scouring action exerted by ice as it scrapes the sediments during periods in the winter when water is withdrawn from the reservoir. Reduced iron released from sediment has a high immediate oxygen demand, serving to remove oxygen from the water immediately surrounding the disturbed sediments. Reduced organic compounds released at the same time may provide microorganisms in the water with available carbon sources that can be used to support additional oxygen-consuming activities within the oxygen sag.

Sediments in the downstream areas that are deeper and less likely to be disturbed may serve as diffusional sources of reduced chemicals creating an oxygen demand in bottom waters near the dam. This area periodically experiences localized pockets of anoxic water that build from the bottom upwards, as described in Gunnison and Barko (1988). In particular, the high levels of ammonium-nitrogen present in sediments in the vicinity of miles 1.0 to 3.0 may contribute specifically to the winter hypolimnetic oxygen demand.

## FUTURE STUDIES

Additional studies will continue in an effort to verify the above scenario. During 1990, one sampling trip will made in late fall to early winter to determine whether sediments in the area of miles 10 to 12 have continued to accumulate large levels of oxygen-demanding substances. A second trip will be made in late winter to early spring following drawdown, to determine if sediment displacement has occurred. This information will be coupled with detailed information on DO sag formation and movement in 1990.

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# EFFECTS OF NET PEN AQUACULTURE ON LAKE WATER QUALITY

by

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and John Veenstra[2], Carlos Ruiz[3]

## INTRODUCTION

Despite an increasing interest in aquaculture operations in the United States, few studies have adequately characterized the waste associated with these activities or assessed impacts of such operations on ambient water quality. A thorough understanding of these impacts is essential to protection and effective management of multi-use waters supporting aquaculture facilities. This paper presents results of a three-year study conducted by the Tulsa District, U.S. Army Corps of Engineers (COE) aimed at assessing water quality-related impacts of aquaculture of channel catfish (Ictalurus punctatus) in floating net pens at Lake Texoma, Oklahoma/Texas.

On 14 April 1987, an aquaculture research demonstration project was approved by the Assistant Secretary of the Army for implementation at Lake Texoma. The purpose of the project was to test the technical, operational, and economic feasibility of using Federal waters for commercial aquaculture. RedArk Development Authority (RedArk), a public trust and economic development organization serving a 21-county area of east-central and south-eastern Oklahoma, was licensed to construct and operate a net pen facility in the Rock Creek Arm of Lake Texoma for annual production and commercial sale of approximately 250,000 channel catfish. In addition to RedArk and the COE, agencies cooperating in the demonstration project included the Soil Conservation Service (SCS), U.S. Fish and Wildlife Service (USFWS), and Agriculture Research Service (ARS).

## DESCRIPTION OF STUDY AREA

Lake Texoma is a 36,032 ha (89,000 acre) impoundment located at river mile 725.3 on the Red River. Completed in 1944 by the U.S. Army Corps of Engineers, the lake occupies portions of both south-central Oklahoma and north-central Texas (Figure 1). At normal pool, maximum depth of the lake is 34 m (112 ft) and mean depth approximately 9 m (30 ft). General water quality is characterized by moderate to high levels of mineralization with a predominance of sodium and calcium salts of sulfates and chlorides (Leifeste et al. 1971).

Net pen facilities were located in the Rock Creek Arm of Lake Texoma adjacent to Platter Flats public use area (Figure 1). The site was selected following evaluation of potential sites on seven COE lakes within the boundaries of RedArk's Trust area (RedArk 1986). Rock Creek Arm covers a surface area of approximately 4 km$^2$ (1.5 miles$^2$), is 4.2 km (2.6 miles) in length, and has an average width of 0.8 km (0.5 miles). Net pens were anchored in an open water location approximately 100 m (330 ft) offshore to allow maximum water exchange through the pens. At normal lake level, water depth beneath the pens was approximately 12 to 15 m (39 to 49 ft).

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[1]Tulsa District U.S. Army Corps of Engineers

[2]Oklahoma State University

[3]Environmental Laboratory, U.S. Army Engineer Waterways Experiment Station

FIGURE 1. Location of Lake Texoma, Rock Creek Arm, and water quality sampling sites.
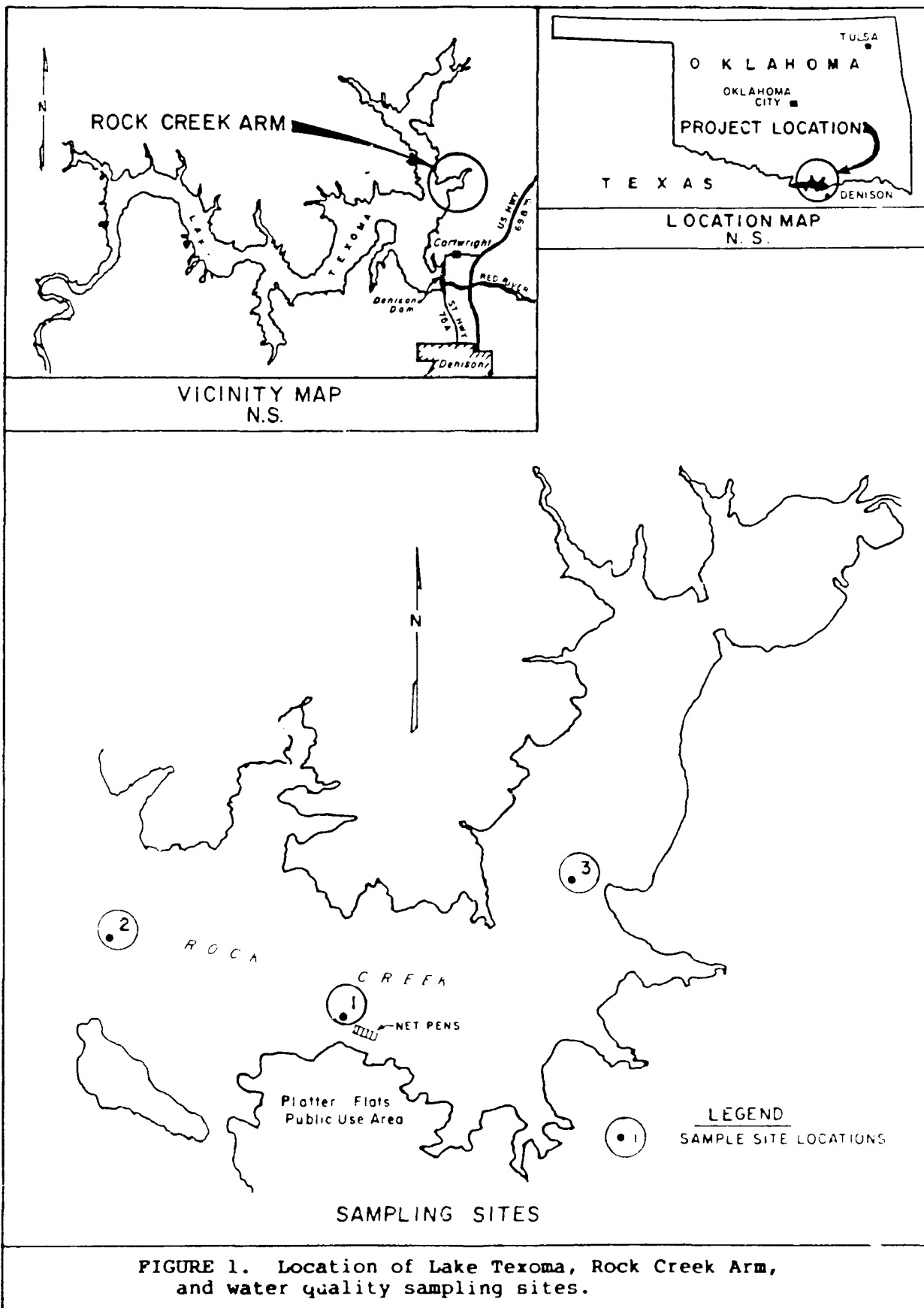
## DESCRIPTION OF NET PEN FACILITIES AND OPERATIONS

Culture facilities were patterned after net pens used for salmonid aquaculture in coastal regions throughout the world. Net pens consisted of five large nylon nets measuring 12.2 m (40 ft) by 6.1 m (20 ft) with a depth of approximately 6.1 m (15 ft) suspended from a floating framework of galvanized metal, wood decking, and styrofoam flotation. Nets were constructed of 2 cm (3/4 inch) square mesh nylon twine treated with a commercial net preservative. The entire structure was securely anchored offshore with cable length sufficient for changes in lake surface elevation. Fish stocking densities in each pen varied from 2.5 to 6.2 fish/ft$^3$ over the three-year study.

Wide variations in lake surface elevation occurred at Lake Texoma over the study period. Flood conditions persisted throughout most of the summers of 1987 and 1989. In contrast, south-central Oklahoma experienced drought conditions during the summer of 1988. While rainfall amounts were slightly lower than normal, 1988 conditions were generally typical of "normal" summer hydrologic conditions at Lake Texoma.

## MATERIALS AND METHODS

Water quality data were collected at three sampling sites within the Rock Creek Arm (Figure 1). Site 1 was located immediately adjacent to the net pen facilities, Site 2 in the mouth of Rock Creek Arm, and Site 3 on the east side of Rock Creek Arm northeast of the pens. Site 3 was chosen as a "control" site due to its morphometric similarities to the net pen site and the anticipated movement of water toward the main body of the lake during hydropower generation and flood releases. Validity of this assumption was supported by initial flow measurements and results of solids traps experiments (Tulsa District USACE, 1989). Data collected at Site 2 were used in evaluating the possible movement of materials resulting from aquaculture activities from the pens toward the main body of the lake. Sampling sites outside of the Rock Creek area were considered but ultimately avoided due to significant horizontal variations in lake water quality and logistic concerns over anticipated rough water conditions outside the arm.

Water quality sampling was initiated on 5 May 1987. Sampling generally proceeded biweekly April through September and monthly October through March throughout the remainder of the study. In all, the study incorporated 38 sampling dates. Field measurements of water temperature, pH, dissolved oxygen, and conductivity were recorded at 1 m depth intervals and water samples were collected at a depth of 0.5 m (1.6 ft) and near the bottom of the water column (generally 10 to 12 m) at each site on all sampling dates. Secchi disc transparency was also measured at each site.

Water sample analyses included the following parameters: total alkalinity, total hardness, nephelometric turbidity, chloride, sulfate ($SO_4$), orthophosphate, total phosphorus, nitrate-N ($NO_3$-N), nitrite-N ($NO_2$-N), ammonia-N ($NH_3$-N), total Kjeldahl nitrogen (TKN), total organic carbon (TOC), dissolved organic carbon (DOC), biochemical oxygen demand (BOD) and chlorophyll a (surface samples only). Analyses were performed according to methods prescribed by the U.S. Environmental Protection Agency (EPA 1983). Chlorophyll a concentrations (not corrected for phaeopigments) were determined fluorometrically (ASTM 1979).

Other sampling activities associated with the project included laboratory and in-situ measurements of sediment oxygen demand (SOD), sediment analyses, and solids traps experiments. Results of these analyses are not presented here but are contained in the final project report (Tulsa District USACE, 1989).

While a number of approaches to data analysis for this study are conceivable, data analysis procedures were based on hydrologic conditions and operational regimes. Originally, only data collected during the fish growing season (basically April through September) were to be used in determining impacts of net pen operations on lake water quality among stations with winter data used as routine monitoring in the absence of aquaculture activities or to test for residual effects following fish

harvest. However, due to the prolonged harvest schedule (and fish overwintering during 1987), there was seldom a period when at least some fish were not residing in the pens. Therefore, all data collected during the study (including winter data) were analyzed collectively. Despite this approach, growing season data were of increased importance in the analysis due to more frequent sampling during these months.

Due to widely varying hydrologic conditions during the study, two approaches to data analysis were used for parameters of particular importance to the study. It is questionable that flood conditions experienced during the early summers of 1987 and 1989 represent lake conditions during a "typical" summer at Lake Texoma (or similar Tulsa District lakes). Therefore, in addition to collective analysis of data from the entire study, data for selected parameters from the 1988 growing season were analyzed separately in an attempt to possibly document water quality effects during a reasonably "normal" year.

Statistical analyses were conducted using Statistical Analysis Systems, Inc. (SAS) programs. Analysis of variance was performed using the General Linear Models (GLM) procedure and Duncan's multiple range test used in means comparisons. A significance level of 0.05 was used throughout the study.


STUDY RESULTS

Field Parameters

Water temperatures followed seasonal and spatial trends common to midwestern impoundments. Isothermal conditions were generally observed at all sites during fall, winter, and early spring months with thermal stratification developing during the summer and generally persisting to early or mid-September. Over the entire study, mean water temperature (including all depths) at Site 1 was 0.6 to 0.8 degrees C lower than means at Sites 2 and 3, respectively (Table 1). Tests for statistically significant differences in mean water temperatures conducted on a depth-specific basis (1 m intervals) yielded unanticipated results. For all data, mean water temperatures among sites at 0.5 and 1 m were not significantly different. However, mean temperatures at all depths between 2 and 8 m at Site 1 were significantly lower than those at Sites 2 and 3 (Figure 2). At 9 and 10 m, mean temperatures were highest at Site 3 with those at Sites 1 and 2 not significantly different. For each depth below 10 m, temperature means at all sites were significantly different.

For 1988 data subjected to the same statistical analyses, mean water temperatures at all depths from 2 to 10 m were not significantly different at Sites 1 and 2 but significantly higher at Site 3. Reasons for slightly decreased water temperatures in the vicinity of the net pens are unknown.

Field pH readings over the entire study varied from 6.00 to 8.45 with the greatest range in values observed at Site 1. Due to difficulties in averaging pH values (requirement for conversion to actual hydrogen ion concentrations), mean values were not computed or compared among sampling sites. Instead, individual readings on sampling dates were compared over the entire study. While differences among pH readings at sampling sites on each date were generally slight, differences that were observed were greatest in surface water strata. Readings at 1 m were generally indicative of these pH differences. Field pH readings at 1 m were lowest at Site 1 on 50% of the 38 sampling trips incorporated in the study. Of these 19 trips, 16 occurred during spring or summer with the remaining three conducted during fall months. Field pH readings at 1 m were highest at Site 1 on only four of the 38 total sampling trips. For 1988 sampling dates only, 1 m pH readings were lowest at Site 1 on seven of 15 trips (47%) with all but one of these seven trips conducted during spring or summer. Readings were lowest at Sites 2 and 3, five times each, over the entire study with fairly even distribution of these dates among seasons.

Conductivity readings were high relative to those recorded at other Tulsa District lakes and generally increased appreciably with depth at all sampling sites. Mean conductivity readings (including all depths) were highest at Site 2 (mean = 1310 uS/cm) and lowest at Site 3 (mean = 1278 uS/cm)

Table 1. Means and standard deviations (SD) for water quality parameters, Lake Texoma. Values include all sampling dates and water depths and are expressed as mg/l unless otherwise noted.

| Parameter | Site 1 Mean | SD | Site 2 Mean | SD | Site 3 Mean | SD |
|---|---|---|---|---|---|---|
| Water Temperature (°C) | 20.80 | 7.29 | 21.40 | 6.54 | 21.59 | 7.24 |
| Dissolved Oxygen | 6.79 | 3.63 | 6.96 | 3.49 | 7.28 | 3.57 |
| Conductivity (uS/cm) | 1304 | 294 | 1310 | 279 | 1278 | 304 |
| Total Alkalinity (as CaCO$_3$) | 127 | 13 | 128 | 13 | 127 | 12 |
| Total Hardness (as CaCO$_3$) | 336 | 51 | 341 | 51 | 339 | 50 |
| Turbidity (NTU) | 6.9 | 3.6 | 7.9 | 24.0 | 7.1 | 3.9 |
| Chloride | 220 | 58 | 218 | 62 | 221 | 58 |
| Sulfate | 217.0 | 47.8 | 216.8 | 50.4 | 218.0 | 50.3 |
| Orthophosphate | 0.007 | 0.005 | 0.007 | 0.005 | 0.007 | 0.004 |
| Total Phosphorus (as P) | 0.036 | 0.014 | 0.032 | 0.021 | 0.034 | 0.012 |
| Nitrate - N | 0.138 | 0.199 | 0.156 | 0.229 | 0.153 | 0.226 |
| Nitrite - N | 0.014 | 0.018 | 0.011 | 0.008 | 0.010 | 0.006 |
| Ammonia - N | 0.061 | 0.051 | 0.059 | 0.053 | 0.053 | 0.041 |
| Total Kjeldahl Nitrogen | 2.55 | 2.19 | 2.37 | 2.06 | 2.35 | 2.76 |
| Total Organic Carbon | 18 | 8 | 18 | 8 | 18 | 9 |
| Dissolved Organic Carbon | 17 | 8 | 16 | 8 | 17 | 8 |
| Biochemical Oxygen Demand (5-Day) | 1.5 | 1.0 | 1.4 | 0.7 | 1.5 | 0.7 |
| Secchi Transparency (m) | 1.0 | 0.3 | 1.2 | 0.3 | 1.0 | 0.2 |
| Chlorophyll a (ug/l) | 14.15 | 12.02 | 12.69 | 7.41 | 14.34 | 7.75 |

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1987-1989 DATA

MEAN TEMPERATURE (°C)

DEPTH (m)

LEGEND

o    SITE 1
●    SITE 2
▲    SITE 3

At each depth, means
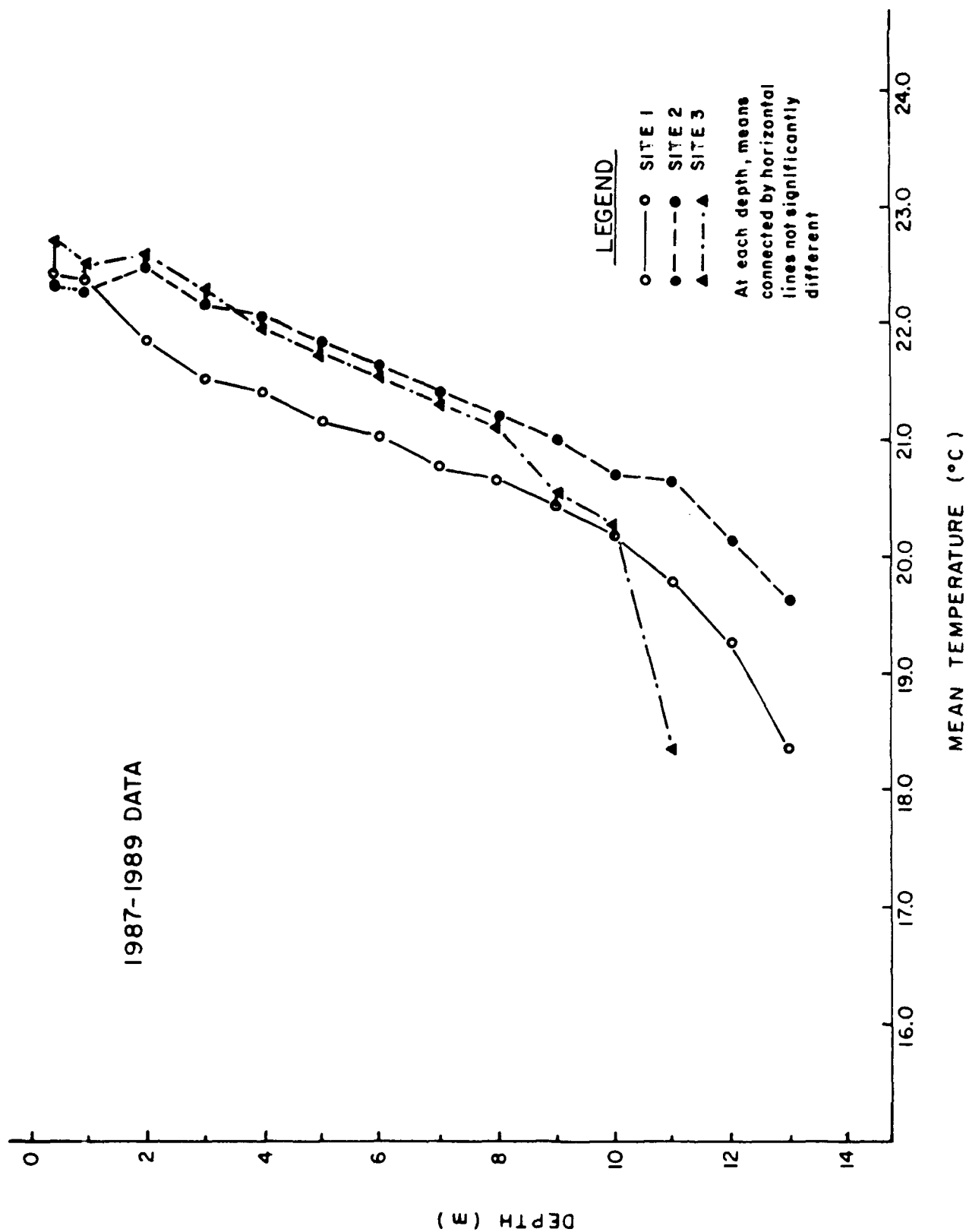connected by horizontal
lines not significantly
different

FIGURE 2. Mean water temperatures, Lake Texoma.

(Table 1). For 1988 data only, similar trends were observed but mean readings were slightly higher at all sites compared to those for the entire study period.

Depth-specific tests (1 m intervals) for statistically significant differences in mean conductivity levels among sites were conducted using data from the entire study (Figure 3). While surface (0.5 and 1 m) means were not significantly different, means at all depths from 3 to 5 m at Site 1 were significantly higher than those at Sites 2 and 3. At 6 m, means at all sites were significantly different. Means at Sites 1 and 2 at all depths from 9 to 12 m were not significantly different from each other but both significantly higher than those at Site 3. Data exclusively from 1988 subjected to the same analyses yielded somewhat different results. While surface means at all sites were not significantly different, means at all depths from 2 to 11 m at Sites 1 and 2 were not significantly different from one another but significantly higher than those at Site 3.

Effects of net pen aquaculture activities on lake dissolved oxygen levels were of particular importance to this study. Therefore, DO data were subjected to a number of different analyses. Mean DO levels (average of readings from all depths) for the entire study period were lowest at Site 1 and highest at Site 3 (means of 6.79 and 7.28 mg/l, respectively) (Table 1). For 1988 data only, similar trends were observed with respect to comparisons among stations although DO means were depressed by 0.13, 0.05, and 0.44 mg/l at Sites 1 through 3, respectively.

Clinograde oxygen profiles were recorded during summer stratification with drastic oxygen reductions occurring at approximately 8 to 10 m (26 to 33 ft) at all sites. In an attempt to define generalized aquaculture-related effects on DO levels in the water column as a whole and in epilimnetic and hypolimnetic layers, DO readings for each sampling date were segregated into surface (< 9 m) and bottom (> 9 m) compartments. Averages were then obtained over the entire water column and for surface and bottom layers (roughly corresponding to epilimnetic and hypolimnetic averages respectively for stratification periods) for all sampling dates (including nonstratification periods). Means of these values were then tested for significant differences among sites.

Using all data, the mean of entire water column averages at Site 1 (6.51 mg/l) was significantly lower than those for Sites 2 and 3 (6.97 and 7.23 mg/l respectively). However, for 1988 data only subjected to an identical analysis, no significant differences among sites were detected (means = 6.56, 6.97, and 6.92 mg/l for Sites 1 through 3, respectively). Means of surface layer averages were significantly higher at Site 2 than those at the other two sites using either data set (means = 7.33, 8.02, 7.59 [all data] and 7.09, 7.98, 7.13 [1988 only] mg/l for Sites 1 through 3, respectively). Increased surface aeration due to the more open water location of Site 2 relative to other sampling sites probably accounts for these differences. Means of bottom water layer averages were all significantly different using all data (means = 4.70, 5.09, 4.28 mg/l for Sites 1 through 3, respectively) and significantly lower at Site 3 (mean = 2.77 mg/l) than at the other two sites (means = 4.98 and 4.91 mg/l for Sites 1 and 2 respectively) using only 1988 data.

As an alternate data analysis method, tests for statistically significant differences in mean dissolved oxygen levels were conducted on a depth-specific (1 m intervals) basis (similar to analyses already presented for temperature and conductivity). Using all data, mean DO concentrations at all depths from 0.5 to 3 m at Sites 2 and 3 were not significantly different from one another but both significantly higher than those at Site 1 (Figure 4). At all depths from 5 to 9 m, mean values were significantly different at all sampling sites. Similar patterns were obtained using 1988 data only, except that significantly lower mean values at Site 1 occurred at 1 and 2 m depths only and mean values at all depths from 7 to 10 m were significantly higher at Sites 1 and 2 than those at Site 3.

One anticipated effect of net pen aquaculture activities on lake water quality was a possible local reduction in DO near the net pens at lake turnover. While these conditions did not develop during 1987, significant DO reductions were observed in the vicinity of the net pens following lake turnover near the end of August 1988 (Figure 5). While DO levels were depressed throughout the Rock Creek Arm at this time, DO concentrations at Site 1 were reduced to levels marginal for fish culture. On 7 September 1988, surface DO concentrations were 7.64 and 6.29 mg/l at Sites 2 and 3, respectively while a reading
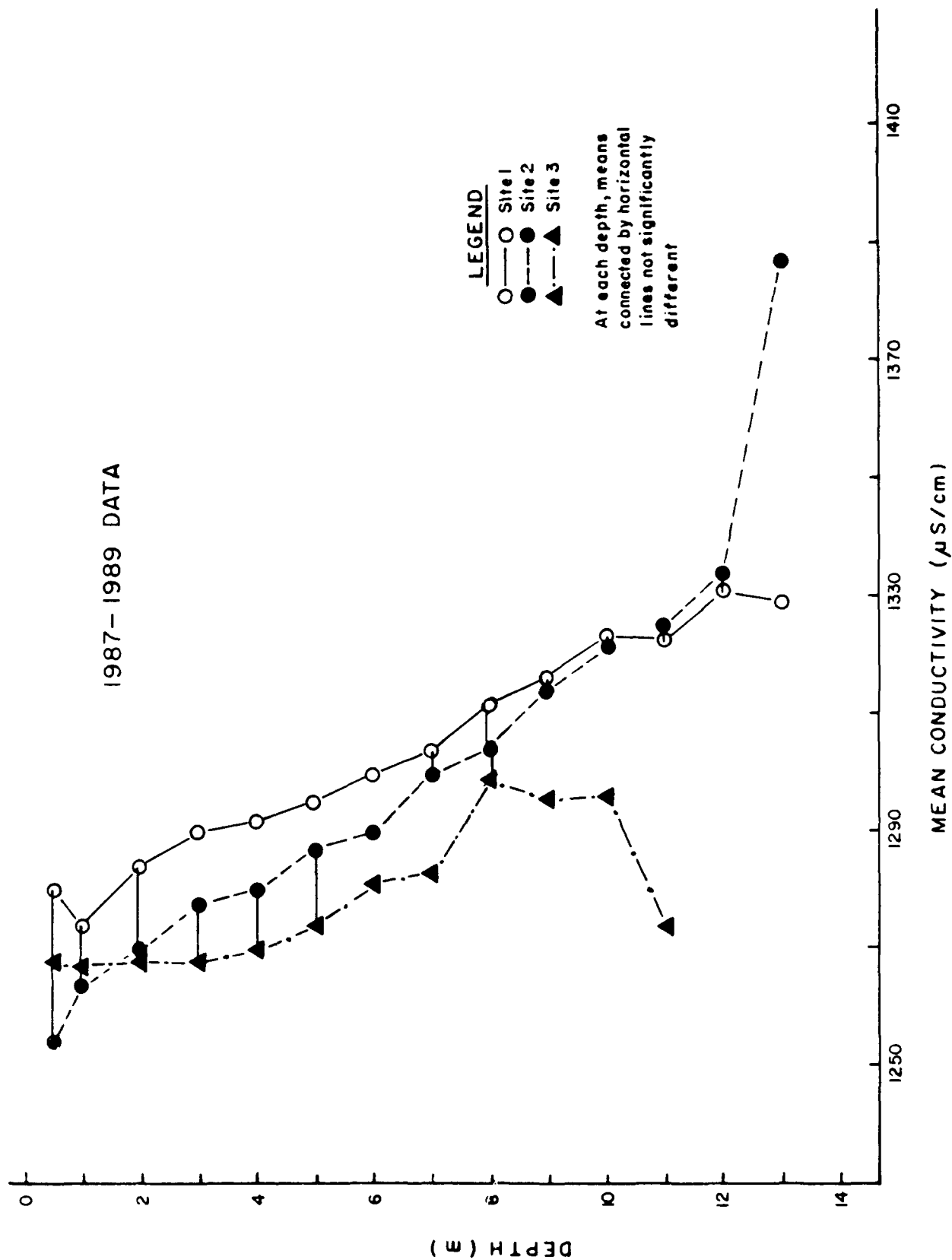
FIGURE 3. Mean field conductivity readings, Lake Texoma.

1987-1989 DATA

LEGEND

○ — SITE 1

● --- SITE 2

▲ —·—·— SITE 3

At each depth, means connected by horizontal lines not significantly different
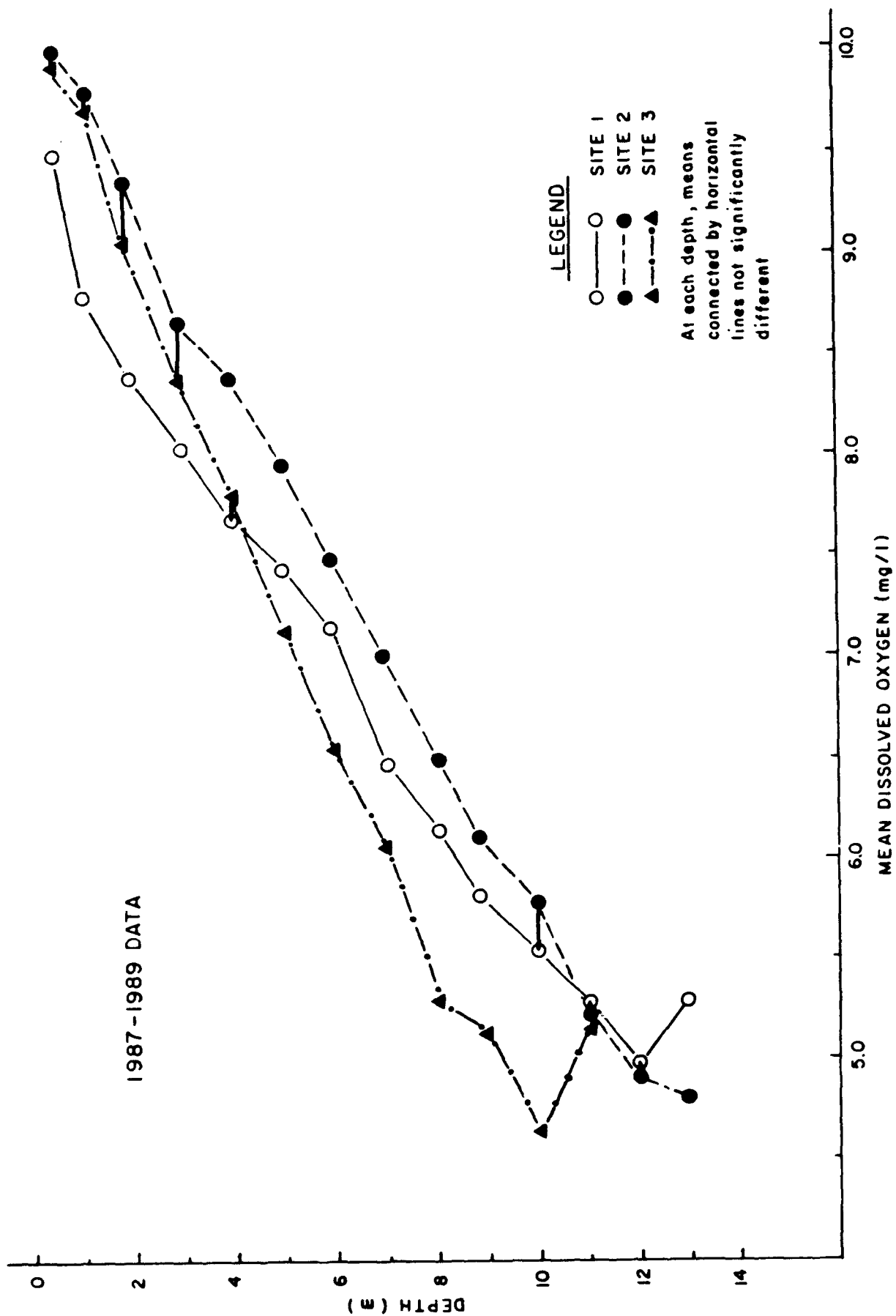
FIGURE 4. Mean dissolved oxygen concentrations, Lake Texoma.
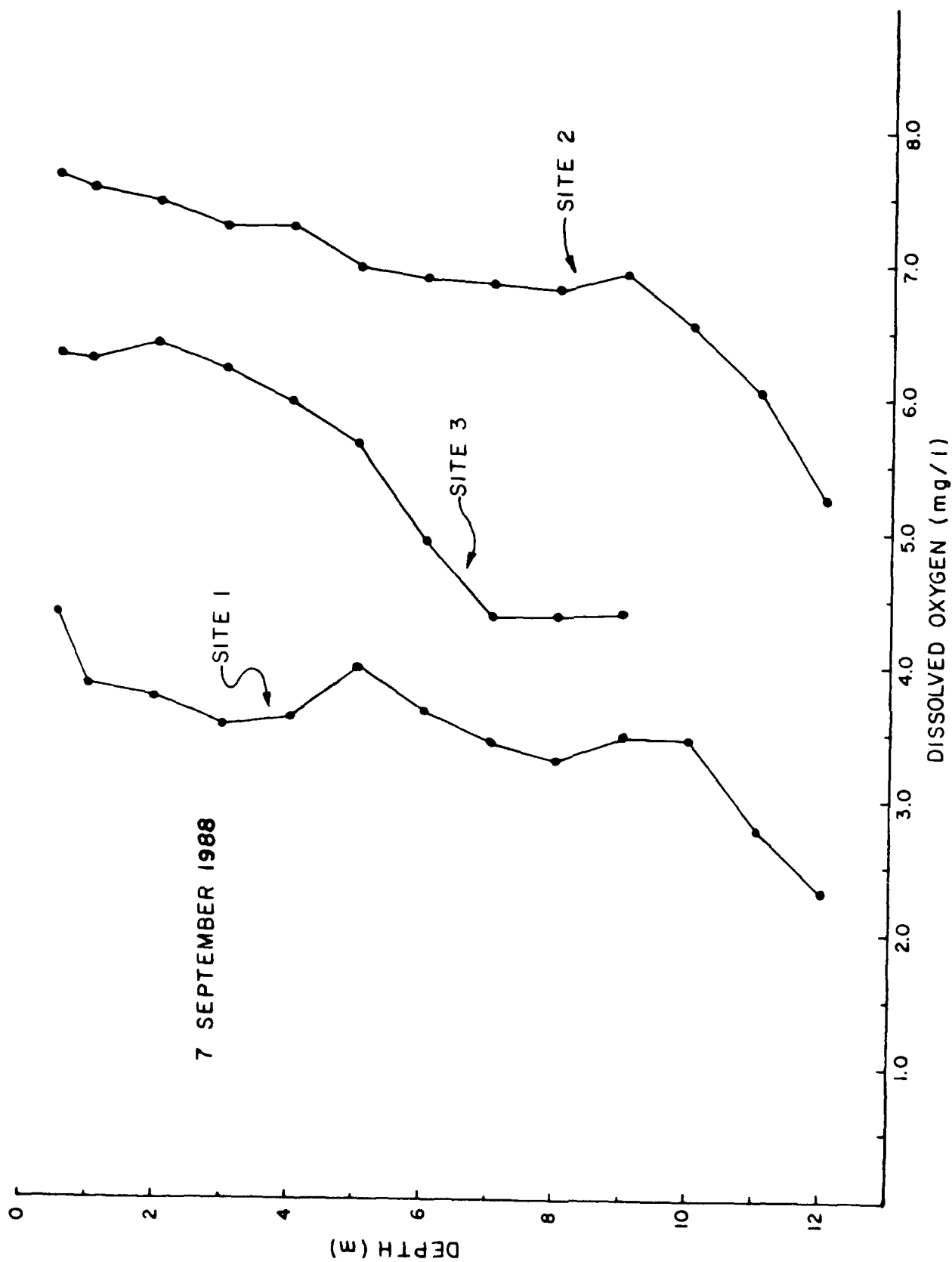
MEAN DISSOLVED OXYGEN (mg/l)

DEPTH (m)

FIGURE 5. Dissolved oxygen profiles, 7 September 1988, Lake Texoma.

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of 4.40 mg/l was recorded at the pens. During this time, fish feeding activity was significantly reduced and an abundance of waste feed was observed floating in the pens. Relative contributions of overfeeding and lake turnover to oxygen depression near the pens were unknown. By 19 September 1988, DO concentrations at the net pens were again similar to those measured at other sampling sites. Data collection ceased prior to lake turnover in 1989.

Secchi disc transparency measurements ranged from 0.5 m, recorded on several dates at Site 1, to a maximum of 1.9 m at Site 2 on 6 July 1988. Over the entire study, mean Secchi transparency was highest at Site 2 (mean = 1.2 m) and identical at Sites 1 and 3 (means = 1.0 m) (Table 1). For 1988 data only, mean values were slightly increased to 1.1, 1.2, and 1.0 m at Sites 1 through 3, respectively. Mean values were significantly highest at Site 2 for both time periods.

For all data, significant correlations were observed between Secchi transparency and chlorophyll $\underline{a}$ at all sampling sites but were slightly closer at Site 2 ($r=-0.68$, $p<0.0001$, $n=30$) than at Site 1 ($r=-0.53$, $p=0.003$, $n=29$) or Site 3 ($r=-0.47$, $p=0.009$, $n=29$). Using 1988 data only, correlation coefficients for these parameters were greatly increased at Site 1 ($r=-0.77$, $p=0.003$, $n=12$), likewise increased at Site 3 ($r=-0.65$, $p=0.028$, $n=12$), and relatively unchanged at Site 2 ($r=-0.67$, $p=0.017$, $n=12$). Interestingly, significant correlations were observed between Secchi transparency and nephelometric turbidity at Site 1 ($r=-0.68$, $p<0.0001$, $n=33$ and $r=-0.82$, $p=0.002$, $n=11$ for all and 1988 data respectively) but correlations were not significant for these parameters at Site 2 ($r=0.24$, $p=0.171$, $n=33$ and $r=0.33$, $p=0.319$, $n=11$ for all and 1988 data respectively) or Site 3 ($r=-0.04$, $p=0.848$, $n=32$ and $r=0.04$, $p=895$, $n=10$ for all and 1988 data respectively).

Laboratory Analyses

Overall means and standard deviations for all water quality parameters at each sampling site are presented in Table 1. Little variation was observed among sites in total alkalinity, total hardness, chlorides or sulfates for any water depth or time period. Based on these analyses, waters in the vicinity of the Lake Texoma net pen site can be classified as very hard, highly mineralized, and well-buffered against drastic pH shifts.

Over the entire study, mean nephelometric turbidity was highest at Site 2 (7.9 NTU) and lowest at Site 1 (6.9 NTU) (Table 1). Mean values obtained from 1988 samples only were similar to those computed for the entire study. Using all data, turbidity means in bottom water samples at Site 2 were significantly lower than those at Sites 1 and 3. Significant differences were not identified in surface turbidity means over the entire study or in means from surface or bottom samples collected exclusively during 1988.

Due to the frequent role of phosphorus as the nutrient limiting growth of algal populations in lakes, effects of net pen aquaculture on phosphorus dynamics were of particular importance to this study. Orthophosphate concentrations were generally below analytical detection limits at all sites during spring and summer months and mean concentrations over the entire study varied little among sites (Table 1). Over the entire study, mean values from analysis of bottom water samples were slightly higher (by 0.001 to 0.003 mg/l) than those for surface samples. Mean orthophosphate concentrations computed from 1988 data were very similar to those for the entire study period. Using all data, means for orthophosphate concentrations in bottom samples at Sites 1 and 2 (mean = 0.009 mg/l at both sites) were significantly higher than the mean at Site 3 (0.007 mg/l). Statistically significant differences in orthophosphate means in surface samples over the entire study or in those from either water depth for 1988 data only were not detected.

Total phosphorus concentrations ranged from 0.005 to 0.171 mg/l (as P) and averaged 0.034 mg/l over the entire study. Mean values for all samples (both depths) were highest at Site 1 (mean = 0.036 mg/l) and lowest at Site 2 (mean = 0.032 mg/l) (Table 1). For surface samples only, total phosphorus means computed over the entire study were identical at Sites 1 and 3 and highest at Site 3 for 1988 data only. For both time periods, total phosphorus means in surface samples were significantly higher at Sites 1 and 3 than those at Site 2 (Figure 6). In bottom water samples, Site 1 possessed the
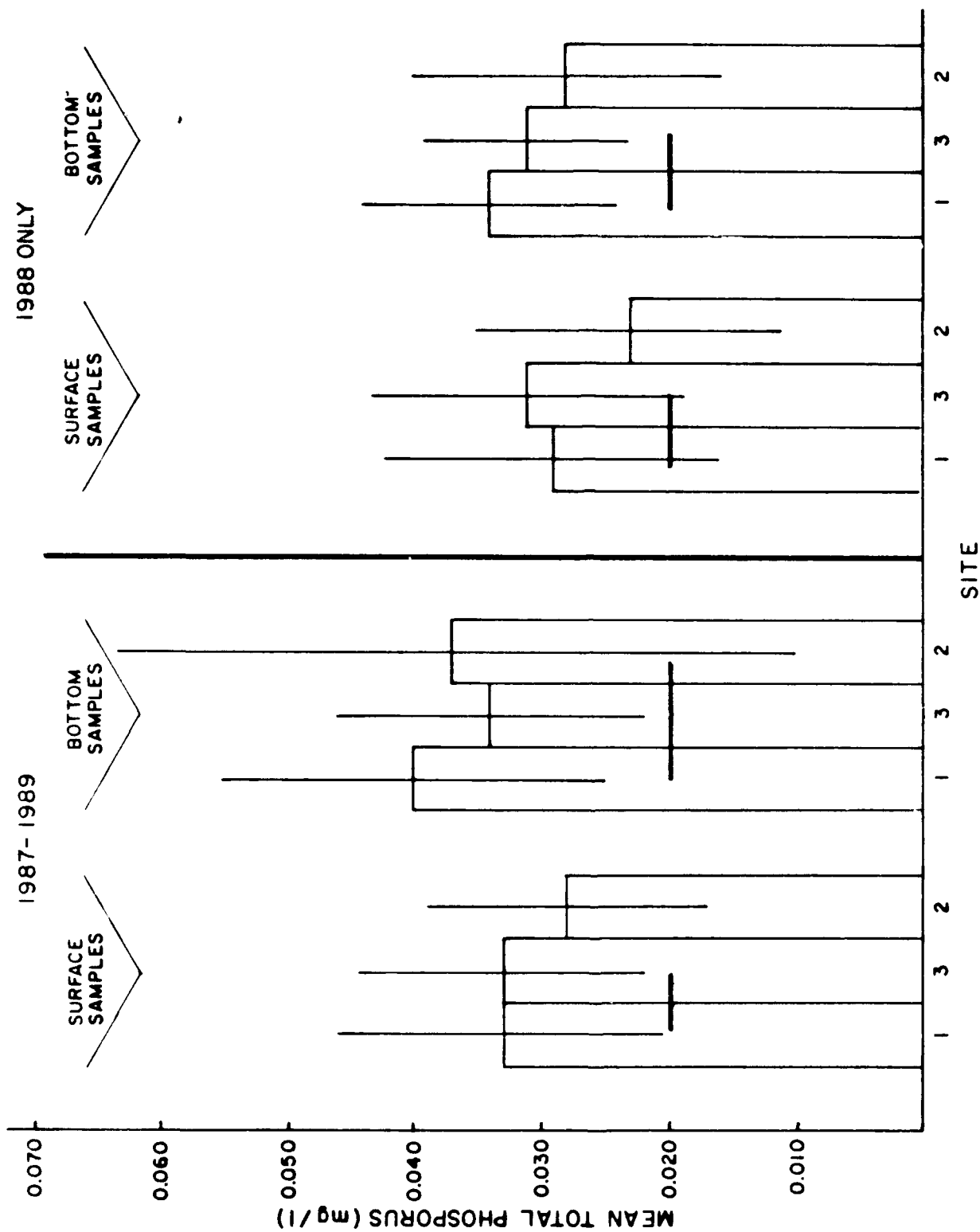
FIGURE 6. Mean (± 1 standard deviation) total phosphorus in
surface and bottom water samples, Lake Texoma. Horizontal
lines connecting bars indicate no significant difference
in means.

highest mean total phosphorus concentration for all data but significant differences among site means were not identified. For bottom samples collected exclusively during 1988, the highest mean total phosphorus concentration occurred at Site 3 and the mean values at this site and Site 1 were significantly higher than mean total phosphorus at Site 2.

Nitrogen parameters were of particular importance to this study due to the excretion of nitrogenous waste products by fish (Lagler et al. 1962) and the ability of nitrogen to stimulate algal production in some aquatic systems (Wetzel 1975). A nitrogen form of particular concern to aquaculturists is ammonia-N ($NH_3$-N). Ammonia concentrations for this study ranged from 0.008 to 0.320 mg/l with a grand mean of 0.058 mg/l. Site-specific overall means ranged from 0.053 mg/l at Site 3 to 0.061 mg/l at Site 1 (Table 1). Over the entire study, mean ammonia concentration was highest in surface water samples at Site 2 (0.065 mg/l) and highest in bottom samples at Site 1 (0.060 mg/l) For 1988 data only, mean ammonia concentrations were slightly higher than those computed over the entire study. Statistically significant differences in ammonia means did not exist among sites at any sampling depth for any analysis time period.

Overall mean nitrite ($NO_2$-N) concentration was highest at sampling Site 1 (0.014 mg/l) and lowest at Site 3 (0.010 mg/l) (Table 1). Over the entire study, mean concentrations were considerably higher for bottom water samples than surface samples at Site 1 but similar at the two depths at Sites 2 and 3. Means for 1968 data only were similar to those for the entire study. In surface samples, mean nitrite concentrations for all data were significantly different at Sites 2 and 3 but neither significantly different from the surface mean at Site 1. Over the same time period, mean nitrite concentration in bottom water samples at Site 1 was significantly higher than those for Sites 2 and 3. Significant differences were not observed in nitrite means in surface or bottom samples collected exclusively during 1988.

Nitrate ($NO_3$-N) concentrations averaged 0.149 mg/l over all sites and sampling dates and ranged from <0.002 to 1.200 mg/l. The highest overall mean nitrate concentration existed at Site 2 (0.156 mg/l) and the lowest at Site 1 (0.138 mg/l) (Table 1). Overall means computed for 1988 data only were considerably higher (by 55%, 62%, and 54% for Sites 1 through 3, respectively) than those for the entire study. For both time periods, mean nitrate concentrations were generally slightly higher in surface samples than in samples collected near the bottom of the water column. Statistically significant differences in mean nitrate concentrations among sampling sites did not exist at any sampling depth for either analysis time period.

Total Kjeldahl nitrogen (TKN) concentrations ranged from 0.10 mg/l at 10 m on 6 July 1989 at Site 2 to 20.25 mg/l at 0.5 m on 16 August 1988 at Site 3. Overall mean TKN concentration over the entire study was highest at Site 1 (2.55 mg/l) and lowest at Site 3 (2.35 mg/l) (Table 1). Overall TKN means for 1988 data were slightly elevated relative to those for the entire study period. While statistically significant differences in mean TKN levels were not observed in surface or bottom samples for all or exclusively 1988 data, mean concentrations were highest at Site 1 in surface samples over the entire study and in bottom samples over both analysis periods. For 1988 data only, mean TKN was highest in surface samples at Site 3.

Over the entire study period, very little variation was observed among sites or sampling depths in total organic carbon (TOC) or dissolved organic carbon (DOC) concentrations. For all data, overall means for TOC were identical (18 mg/l) at all three sampling sites (Table 1). While 1988 means were slightly higher (by approximately 6 mg/l) than those for the whole study period, values were again nearly identical at all sites.

Biochemical oxygen demand (BOD) values varied from 0 to 5.6 mg/l over the entire study with a grand mean of 1.5 mg/l. Site-specific overall means were highest at Sites 1 and 3 (means for both sites = 1.5 mg/l) and only slightly lower at Site 2 (mean = 1.4 mg/l) (Table 1). Over the entire study, BOD values in surface samples were higher than those from bottom samples by an average of 0.7 mg/l. Mean values computed exclusively from 1988 data exceeded those over the entire sampling period by

0.1 to 0.2 mg/l. Statistically significant differences in BOD means were not observed among sampling sites in surface or bottom water samples for either of the analysis time periods.

An important objective of this study was to determine the effects of aquaculture activities on algal dynamics and overall productivity in the vicinity of the net pen facilities. While not a true measure of algal biomass, chlorophyll a concentrations are frequently used as a relative measure of algal abundance. Over the entire study, chlorophyll a values (at 0.5 m) ranged from 2.19 ug/l at Site 2 on 2 May 1989 to 69.85 ug/l on 15 June 1987 at Site 1. Overall site-specific means were 14.15, 12.69, and 14.34 ug/l at Sites 1 through 3 respectively (Table 1). Means computed for 1988 chlorophyll a data only were slightly lower than those incorporating all sampling dates. Over the complete study, mean chlorophyll a concentrations were not significantly different among sites. For 1988 data only, the mean concentration of this pigment at Site 3 was significantly higher than those at Sites 1 and 2. While net pen fouling by attached algae was a constant problem at the facility, floating algal mats or other evidence of nuisance algal "blooms" were never observed at any site.

## DISCUSSION

In general, the most measurable effects of net pen aquaculture on lake water quality identified by this study were slight alterations in commonly-measured limnological field parameters in surface waters near the aquaculture facilities. While statistically significant decreases in water temperature and dissolved oxygen and significant increases in field conductivity were observed in surface waters near the net pens relative to other sampling sites, actual magnitude of these changes was slight and probable impacts on the biology and limnology of surrounding waters were negligible.

Probably the most dramatic water quality effect observed during the study was a decrease in dissolved oxygen levels near the pens following lake turnover in 1988. These effects were not as pronounced during the high water year of 1987. While localized, temporarily depressed DO levels are of major concern to fish culturists, these effects are less significant to overall lake biology due to the ability of native organisms to escape from and avoid stressful conditions. Nevertheless, these effects should be carefully considered and monitored in future net pen aquaculture activities.

Problems associated with nutrients and increased algal production resulting from net pen fish culture were not documented in this study. While aquaculture activities generate a considerable amount of nutrient wastes, water exchange through the net pens at the Lake Texoma site was apparently sufficient to reduce nutrient impacts through pollutant dispersion and natural assimilative processes.

Comparison of results of this study with other aquaculture-related water quality investigations illustrate the importance of sitting fish culture facilities in large, open waters with maximum water exchange and depth. Other investigators have reported numerous water quality effects from facilities located in small, shallow lakes or bays. Eley et al. (1972) reported significant increases in turbidity, alkalinity, total phosphorus, organic nitrogen, BOD, and bacteria and significant decreases in dissolved oxygen associated with cage catfish culture in White Oak Lake, Arkansas. Mean and maximum depths of this small lake were 2.4 m and 5.5 m, respectively. Trojanowski et al. (1987) documented significant increases in chlorophyll a near trout cages in two small, shallow (mean depths of 3.6 and 8.2 m) Polish lakes.

Results of this study were similar to those obtained at Bull Shoals Lake in north-central Arkansas (Hays 1980). Net pen facilities similar to those used at Lake Texoma are located in a large, 81 surface ha (200 acre) cove in Bull Shoals Lake where water depths average 20 m (65 ft). Approximately 200,000 rainbow trout and 250,000 channel catfish are grown to a catchable size each year in these net pens. Results of water quality sampling conducted monthly from 1 October 1978 to 31 January 1980 near the Bull Shoals facility indicated increased levels of ammonia, total phosphate and plankton abundance associated with fish culture activities. Significant changes in a number of other water quality parameters were not observed.

Con,iderable caution and common sense should be exercised in using results of this study to predict effects of net pen aquaculture on water quality in lakes possessing physical, chemical, and hydrologic characteristics vastly different from those at Lake Texoma. Lake Texoma water quality is characterized by high levels of mineralization, moderate productivity levels, and a high pH buffering capacity. Effects of net pen aquaculture on water quality in poorly-buffered, soft water lakes of varying trophic states might be more pronounced and considerably different than those measured by this study. Predictions of effects of aquaculture activities on lake water quality should therefore be conducted on a site-specific basis.

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# DEVELOPMENT OF THE CUMBERLAND BASIN RESERVOIR MODEL
## FOR WATER QUALITY CONTROL

Jackson K. Brown[1] and Donald F. Hayes[2]

## Introduction

The purpose of this paper is to describe the efforts of the Nashville District and Waterways Experiment Station to develop a system model of the Cumberland Basin reservoir system. The model will be capable of simulating impacts of proposed operations on the most important project purposes. These purposes include water quality, hydropower, navigation and recreation. The model is essentially a drought model and many of the simplifying assumptions and coefficients used by the model are not valid for high flow conditions.

## Background

The need to develop a means of evaluating impacts of reservoir system operations on project purposes became apparent during the mid-1980's, when precipitation and stream flows were generally below normal. By this time models had been developed which provided the capability to formulate operational schemes to avoid or alleviate potential water quality problems. However, no progress was made in implementing these schemes because they differed from scheduled hydropower operations. Hydropower was considered an authorized purpose for which tangible monetary benefits could be computed, whereas water quality was not an authorized purpose and benefits were intangible. Although requirements for flood control and navigation were satisfied, in effect the projects were operated during the critical summer months primarily as a single-purpose hydropower system. Benefits to water quality and recreation were essentially incidental. The impetus for the Cumberland Basin reservoir model is to develop a means to optimize system operations which will consider impacts on all project purposes.

## Description of Reservoir System

The Cumberland Basin lies in the states of Tennessee and Kentucky and has a drainage area of 18,000 square miles. Within the basin the Nashville District manages a system of nine major dams, all of which produce hydropower. A map showing the locations of the projects is provided as Figure 1. Five of the projects (Laurel, Wolf Creek, Dale

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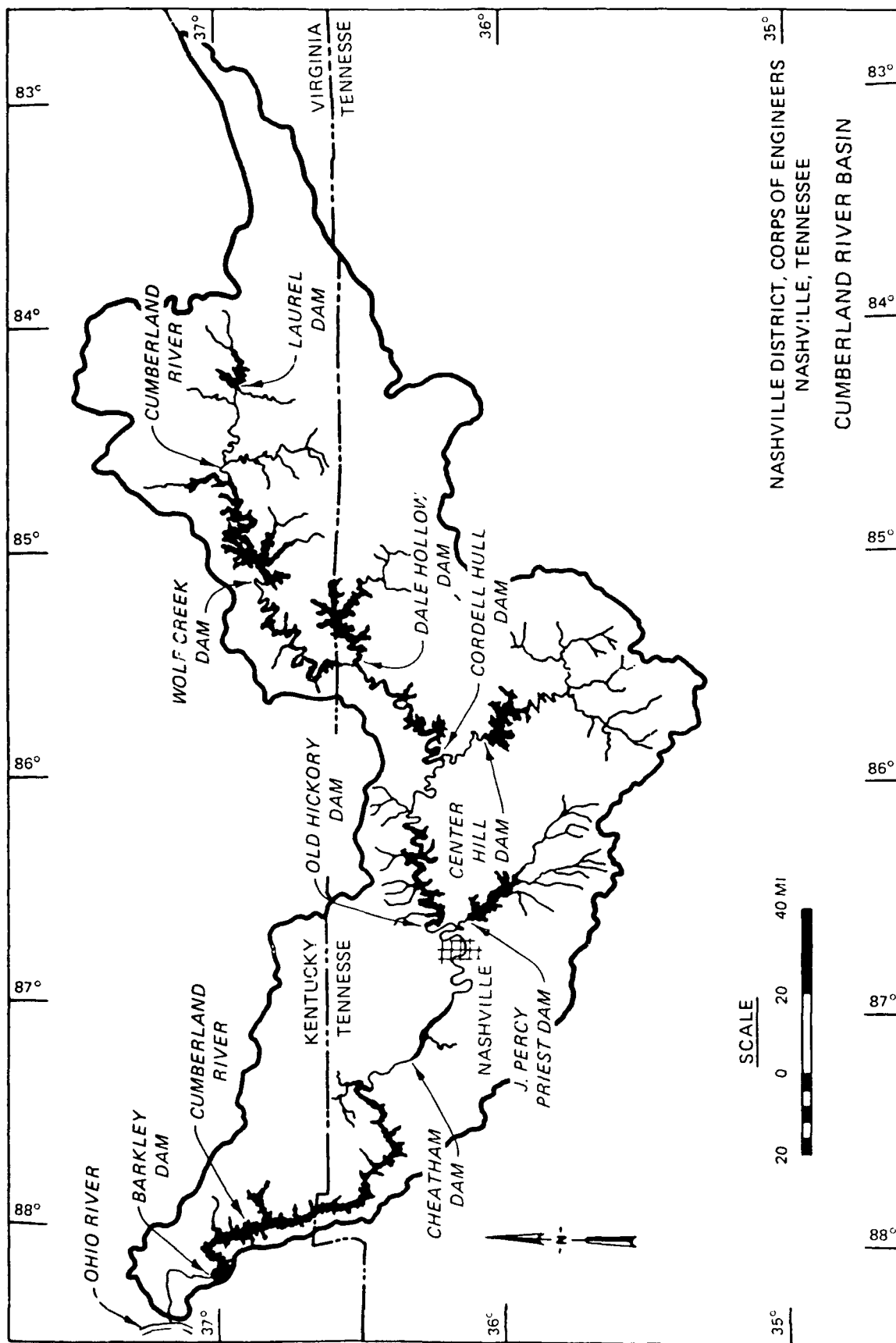
Figure 1. Location map for the Cumberland River Basin

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Hollow, Center Hill and J. Percy Priest) are classified as tributary storage impoundments. The remaining four projects (Cordell Hull, Old Hickory, Cheatham and Barkley) are classified as run-of-river navigation impoundments.

## Water Quality Considerations

The water quality parameter of most concern in the run-of-river projects is dissolved oxygen (D.O.). Releases from the three largest storage impoundments, Wolf Creek, Dale Hollow and Center Hill, are cold and from April through September density currents dominate the hydrodynamic regimes of Cordell Hull and Old Hickory. As the cold inflows travel through these impoundments, density differences prevent mixing with the warm oxygen-rich waters of the euphotic zone and assimilation of organic matter causes D.O. concentrations in the underflows to decrease. The longer a slug of water is detained in the hypolimnion, the lower the D.O. concentration in the hypolimnion and in project releases. Of course, detention time is a function of flows, which are controlled primarily by the regulation of the storage impoundments. Cheatham and Barkley inflows are warmer and density currents are not as dominant. Stratification at these projects is relatively weak and intermittent. Despite the significant organic loading from metropolitan Nashville, reaeration processes in the Cheatham pool generally cause a net increase in D.O. concentrations as water passes through the project. Thus, the critical point for D.O. in the run-of-river projects is the Old Hickory outflow. The operational objective for water quality control is to release sufficient water from the upstream storage projects to maintain D.O. concentrations in the Old Hickory outflows at or above the 5 mg/l state water quality criterion.

## Problem Formulation

It was stated previously that a method is needed to determine the daily releases for these nine reservoir projects which best satisfy all project purposes during drought or low-flow conditions. The ability to explicitly consider water quality responses, particularly D.O., has been deemed especially important. The resulting highly dimensional, multi-objective optimization problem can be stated as

$$\text{MAX } \Phi(S_{t_f}, T_{t_f}, C_{t_f}) + \sum_{t=1}^{t_f} B_t(U_t, S_t, T_t, C_t) \qquad . \qquad (1)$$

subject to

$$S_t = S_{t-1} + R^t U_t + R^{t-1} U_{t-1} + R^{t-2} U_{t-2} + Q_t \qquad (2)$$

$$T_t = f(\overline{T}_{t-\theta_t}, \delta, \theta_t, T_{t-\theta_t}) \qquad (3)$$

$$C_t = \hat{C}(T_t) - D_t \qquad (4)$$

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and

$$S^{min}_t \leq S_t \leq S^{max}_t \qquad\qquad (5)$$

$$U^{min}_t \leq U_t \leq U^{max}_t \qquad\qquad (6)$$

$$T^{min}_t \leq T_t \leq T^{max}_t \qquad\qquad (7)$$

$$C^{min}_t \leq C_t \leq C^{max}_t \qquad\qquad (8)$$

where $\Phi$ = scalar function describing the value of the system state at the final stage $t_f$, $B_t$ = scalar function describing system benefits, $S_t$ = column matrix of reservoir storages at time t, $T_t$ = column matrix of water temperatures at time t, $C_t$ = column matrix of D.O. concentrations at time t, $R$ = flow routing matrix, $Q_t$ = column matrix of local inflows during time t, $\delta$ = temperature rate change coefficient, $\theta_t$ = hydraulic retention time, $C$ = saturation D.O. concentration, $D_t$ = column matrix of D.O. deficits at time t, $S^{min}_t$, $S^{max}_t$, $U^{min}_t$, $U^{max}_t$, $T^{min}_t$, $T^{max}_t$, $C^{min}_t$, and $C^{max}_t$ are lower and upper bounds on the respective variables.

A few years ago, solving such a highly dimensional optimization problem would have been considered impractical particularly with the natural non-convexities introduced by hydropower generation. Hiew (1987), however, found that optimal control theory (OCT) performed rather well for these type problems; OCT was able to reliably find optimal solutions with impressive expediency compared to other algorithms. Details of the application of OCT to the Cumberland River system were presented by Hayes (1989).

The crux of any practical optimization problem is defining the performance index, $B_t$, to accurately reflect the goals and purposes of system operation. Mathematically representing the relative benefits of the many operating variables is quite difficult. To compound the problem, these benefits are often in different, non-commensurate units. This is particularly true of environmental quality benefits such as water quality and various forms of recreation. A simplified approach was presented by Hayes (1989) which essentially ignores "benefits" of all project operations except hydropower, but applies penalties (negative benefits) if water quality or other specified targets are not met. Although this approach is plausible for the Cumberland system, it is by no means a complete solution. Additional terms should be added to the objective function which encourage meeting specific goals.

Description of Water Quality Model

A water quality model named DORMII has been developed to forecast temperature and D.O. conditions in the run-of-river impoundments and determine outflows from the storage projects needed to maintain acceptable water quality conditions. DORMII is essentially a simple 2-D routing model which uses daily computational time steps. For DORMII an impoundment is divided into several reaches having two depths representing the epilimnion and hypolimnion. In the epilimnion the water temperature is

trying to reach the equilibrium temperature and the D.O. is attempting to reach saturation. In the hypolimnion temperatures are increased slightly to account for the penetration of solar radiation and D.O. concentrations are decreased by a bulk depletion rate. One of the first computations the model performs is to determine when the water at the downstream end of the layer in each reach first entered the layer at the upstream end of the reach. These travel time computations for individual layers are based on the volume of the layer and the percentage of flow passing through the layer, which are assumed to be constants. The initial temperature and D.O. values at the upstream end of the reach are routed through each layer and adjusted as they are impacted by the forces of change simulated by the model.

One of the most important phenomena simulated is vertical mixing. The following equation, which was initially formulated to determine plunge points for density currents, is used empirically to determine whether or not vertical mixing occurs at the end of each time step:

$$Z_c = \left[\frac{1}{Fr^2}\right]^{\frac{1}{3}} \left[\frac{U^2}{W_c^2 g \epsilon}\right]^{\frac{1}{3}} \tag{9}$$

where $Z_c$ = critical depth, Fr = effective densimetric Froude number, U = flow rate, $W_c$ = effective width of zone of conveyance, g = acceleration due to gravity, and $\epsilon$ = relative density difference between epilimnion and hypolimnion. If the depth of the reach is less than $Z_c$, complete mixing between the two layers, weighted by the flow in each layer, is assumed to occur. Once water temperature and D.O. are routed through the most downstream reach, withdrawal zone and turbine reaeration factors are applied to compute outflow values.

Model Implementation

The existence of an operational tool such as that which has been described provides several capabilities. By using the model to evaluate historical operations, one can gain insight into operational policies for future conditions which may be similar. The reevaluation of historical operations also provides a method for calibrating relative benefits of reservoir operations such as recreation and water quality. After all, the value of any such optimization model is governed by its ability to simulate decision-maker responses to specific conditions. It is intended, however, for the model to be implemented to assist in real-time operations. Recognizing the restrictions imposed by modelling drought conditions and the use of deterministic optimization, it is anticipated that the model will be applied from an estimated peak headwater time in late spring through the low-flow season when low D.O. levels are common. An initial run, based upon inflow estimates and forecasts of upcoming conditions, near the beginning of this time period would allow a preliminary release plan to be developed. As conditions materialize and differ from forecasts and expectations, additional runs can be made starting with existing conditions to further refine the release plan. Continuation of

this "adaptive planning" methodology allows adjustments for changing and unexpected conditions.

## Acknowledgement

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# POTENTIAL FOR THERMAL STRESS IN JUVENILE CHINOOK SALMON
## AT McNARY DAM, WASHINGTON

by

Edward B. Meyer, CPT[1] and Tim Bartish[2]

## INTRODUCTION

McNary Dam is a multi-purpose Corps of Engineers project located in south-central Washington state on the Columbia River at river mile 292. It was completed in 1954 and presently consists of (oriented from north to south) a navigation lock, a spillway structure with 22 gates (numbered 1 to 22, north to south), and a powerhouse facility with 14 Kaplin turbines (numbered 1 to 14, south to north) and two smaller house units to provide internal power requirements.

Power generation from Lake Wallula upstream is on a run-of-the-river basis and is closely governed by releases from the dams upstream and the flow requirements of the power projects downstream. As such, retention times for inflows are small, typically 2 to 8 days during the summer.[3] Inflow to the lake is provided by the Columbia and Snake Rivers whose confluence is some 31 miles upstream of the dam itself.

McNary Dam has extensive facilities to aid in the collection and transportation of juvenile anadromous fishes. This is done in order to avoid some of the mortality associated with the fish passing through project powerhouses on their way to the sea. The collection system is very similar to other ones in operation at a number of dams on the Columbia and Snake Rivers. Downstream migrating juvenile fish are collected using submerged traveling screens located in the turbine intakes. Each of the 14 turbine intakes is divided into three gates and gatewells, labeled A, B and C, south to north. All of the gatewells are equipped with a trash rack and a submersible traveling screen (STS) which can be raised or lowered into the penstock flow as required for fingerling collection.

At the beginning of the collection season, each STS is deployed into its penstock and angled upstream into the flow. This screen serves to intercept the juvenile fish being swept into the turbine and guide them up into a gatewell located above the ceiling of the penstock (Figure 1). Near the surface of each gatewell are two 12-inch orifices (only one of which is operated at a time) which provide approximately 12-15 cfs of flow to a travel flume which runs across the face of the powerhouse. The fish are swept through this orifice, into the travel flume, and down to the collection facility located between the powerhouse and spillway in the center of the project. Excess water is removed from the flume through a series of screens and perforated plates at regular intervals along the flume wall.

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[3]Tim Bartish. January 1988. "McNary Temperature Study: Thermal Shock," Unpublished data, U.S. Army Engineer District, Walla Walla.
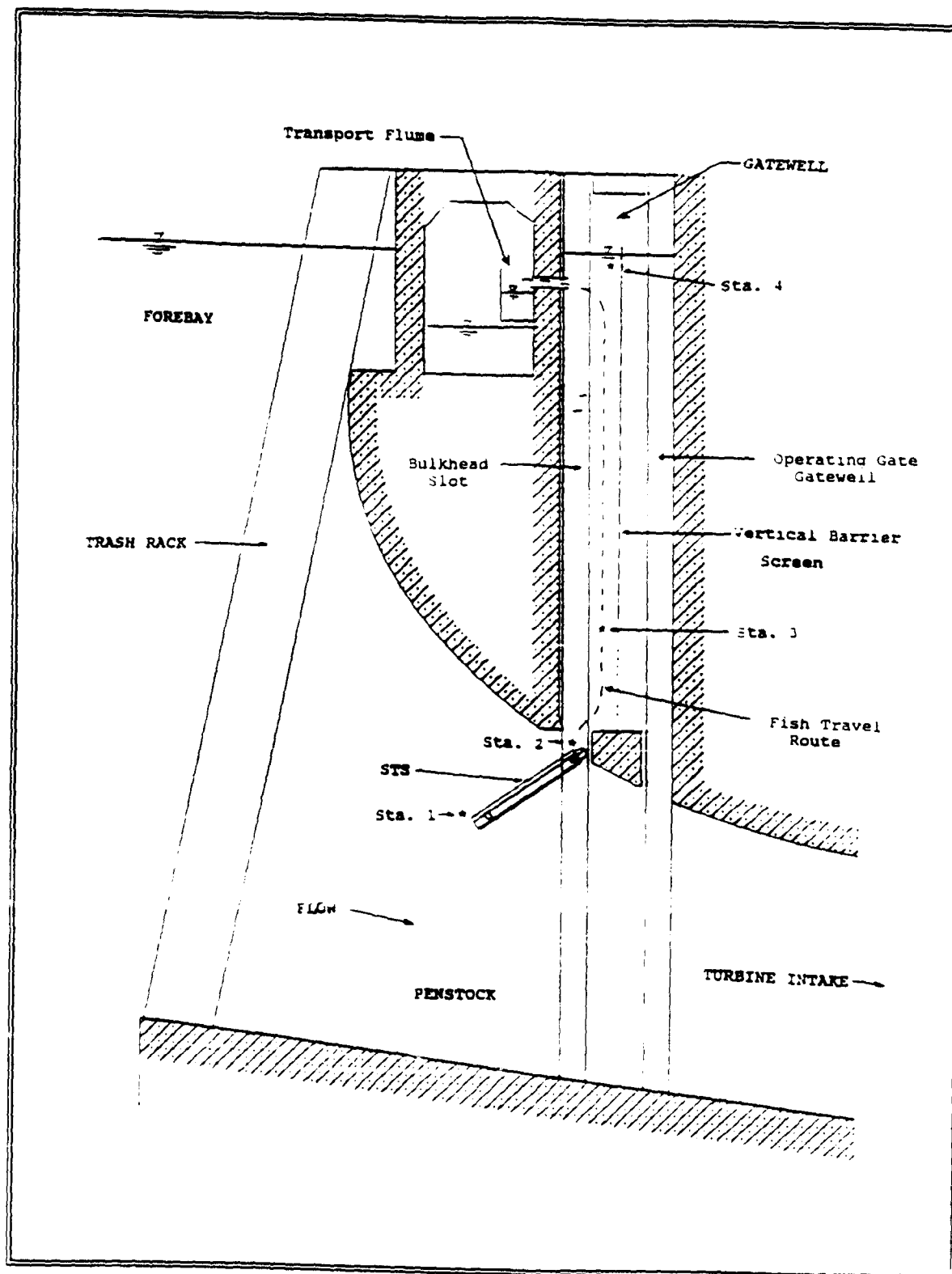
Figure 1. Cross-sectional view of the dam showing the deployment of the submerged traveling screen (STS), the juvenile travel route, and the location of the penstock and gatewell thermistors (Stations 1, 2, 3, and 4).

## CONCERNS AND GOALS

Typical daily mortality at McNary Dam for subyearling fall chinook salmon collected using this system is normally on the order of 1% to 2%[4]. However, during the last four collection seasons, abnormally high daily mortalities (on the order of 10% or greater) have been recorded. It has been theorized by the Walla Walla District (NPW) of the U.S. Army Corps of Engineers that these deaths are directly related to thermal shock to the fish caused by high temperatures and low flow conditions experienced during the summer months. This led NPW to request that personnel of the Waterways Experiment Station (WES) evaluate some of the data collected by NPW during the 1987 and 1988 seasons and provide an assessment of the District's assumption of causes leading to increased mortality and provide recommendations for further data collection or operation changes. In addition, WES was asked to determine if a relationship exists between water temperature fluctuations and collection facility and powerhouse operational conditions. By investigating these areas, modifications to the existing operating plan may be made which could significantly reduce the level of mortalities seen at the collection facility.

A number of different theories on the causes of the high mortality rates seen at the project have been put forth by both NPW and project personnel. Data collected suggest the presence of isolated occurrences, or "blocks", of warm water which are being drawn into the powerhouse causing the thermal stressing of smolts. The source of this warm water is unknown but is thought to be related to conditions occurring in the forebay area or within the powerhouse itself. District and project personnel do not believe that incomplete mixing of the two upstream rivers is the cause of the problem. There appears to be some information that suggests a correlation between project operation and smolt mortality. Water temperature data suggest that the warmer surface water of the forebay is being swept into operating units, that off-units seem to be collecting or attracting fish in the penstock, and that the start up of units are associated with increased thermal stress.

## DATA ANALYSIS

In 1987, NPW data collection and analysis focused primarily on determining if lateral temperature variations between the waters from the Snake and Columbia Rivers affected the temperatures seen at the dam. The investigation found very little evidence of vertical or lateral stratification except in the area just in front and to the south of the dam (Bartish, op. cit.).

In 1988, a much more intensive data collection study was undertaken by NPW to try to determine the source of the warm water in the collection system. During the spring, six recording temperature monitors were installed in the juvenile collection system to examine the effects of operational changes on water temperature. Four of these devices were installed at various depths in the B gatewell of Unit 7 (Figure 1). Unit 7 was selected because it was in the center of the powerhouse and would sometimes be operating depending on the power generation requirements. This allowed data to be collected under operating and non-operating conditions in order to evaluate the effects of powerhouse operation on water temperature.

The remaining two recording units were placed in the transport flume, one upstream of the inflow from Unit 7, and one downstream. This configuration was operated from early June through the end of July. Data collected from July 15 through July 29, 1988 were selected for examination as being representative of dam and powerhouse operations as well as temperature and fish migration patterns. In fact, significant fish mortality occurred during the latter part of this period.

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[4] *Personal communication with Paul Wagner, Fisheries Biologists, Washington State Department of Fisheries - McNary Dam, on 5 May 89.*

Other temperature data collected include several lateral transect and vertical profiles in the forebay area (taken on 25 July 1988) as well as miscellaneous temperature measurements taken by the fisheries people on site at a variety of locations and times. Powerhouse operation records and fish collection and mortality data were also available for the collection season.

The data collected during the forebay temperature survey performed on July 25, 1988 showed various degrees of stratification throughout the forebay. This information provides much insight into the thermal problem. Temperature profiles recorded in front of operating units, and at stations south, show that water is 2°-3°F warmer compared to a profile recorded further out in the forebay except at the surface, which is 1°-2°F cooler. Stations to the north of the powerhouse show only a thin layer of warm surface water. The temperatures recorded from the transect immediately in front of and parallel to the dam shows that the water 4m deep and north of the powerhouse is substantially cooler than that at the same depth and in front and south of the powerhouse. Yet water temperature profiles and transects collected further out in the forebay indicate very little lateral temperature difference in the water column.

These data indicate that operating units are indeed drawing the warm surface water down into the penstocks. As this surface water plunges downward it then begins to mix with the cooler, deeper water. The result is water being drawn across the STS and into the gatewell that is cooler than the surface water in the forebay and yet still significantly warmer than the water found at the same elevation further out in the forebay. The data collected in front of the spillway, which was not spilling, indicate little vertical mixing of the warmer surface waters.

Except for a slight delay, there appears to be no significant temperature difference between the four vertical thermistors in the penstock and gatewell. Fluctuations in the total turbine discharge from the powerhouse have very little influence on the temperatures recorded in the gatewell and penstock. This would indicate that there is no significant source of warm water within the powerhouse.

However, there is a distinct temperature increase or decrease across all of the recorders when Unit 7 starts up or shuts down. This temperature fluctuation is on the order of 2°-3°F over a 15- to 20-minute period. When Unit 7 starts in combination with one or more of the adjacent units (6, 8 or 9), the temperature increase across all of the penstock and gatewell thermistors can be as high as 8°F in a 10- to 15-minute period (Figure 2). This multiple-unit start-up usually resulted in the greatest fluctuations in temperatures over all four thermistors. However, the temperature peak tended to diminish with time, provided no other neighboring units started or stopped operation. Average temperature for Unit 7 at station #4 (just below the water surface in the gatewell), when operating during this 15-day period, was 69.0°F. For non-operating periods, the temperature averaged 65.9°F.

The temperatures in the transport flume can also be dramatically affected by unit operation and the gatewell orifice discharge to the flume. The average temperature of the flume downstream of Unit 7, while Unit 7 was operating, was 68.2°F, and 66.5°F while Unit 7 was idle (with only Unit 1 operating upstream). When a unit is started or stopped, the temperature of the transport flume just downstream of the unit can change very rapidly. Changes of as much as 2° to 4°F over a 30-minute period were recorded during this two week period. Temperature fluctuations in the flume between operating and non-operating units and vice versa can be as high as 4° to 5°F (Figure 3). These changes take place in the flume over the relatively short distance of a single unit.

## MORTALITIES

For the 15-day period being examined here, the average daily mortality was 4.8%. The maximum daily mortality was approximately 21% and occurred on July 26. The maximum hourly mortality for each day during the study period was significantly greater and ranged as high as 67% on July 26. For the majority of the time, however, the hourly mortality was less than 4%, although there were periods of time when this was greatly exceeded. From the data, it would appear that mortalities are closely tied to changes in total powerhouse discharge and thermal shock is a compounding factor.

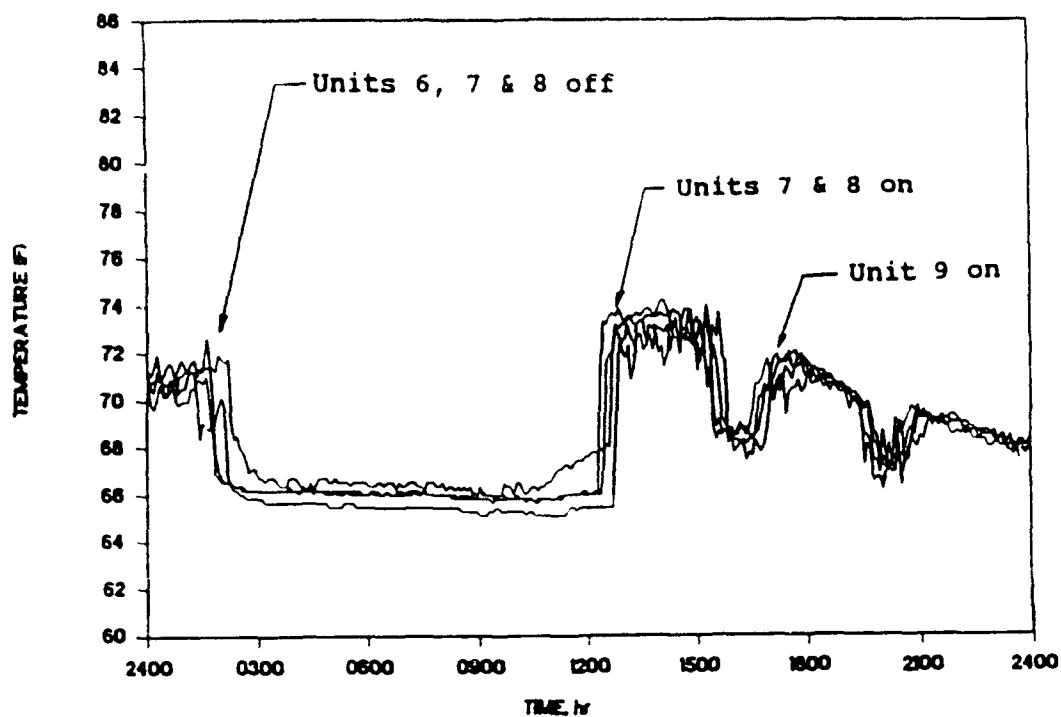Figure 2. Temperatures recorded across the penstock and gatewell thermistors showing the typical affects of unit operations on the temperature recorded in slot 7B (27 July 1988).
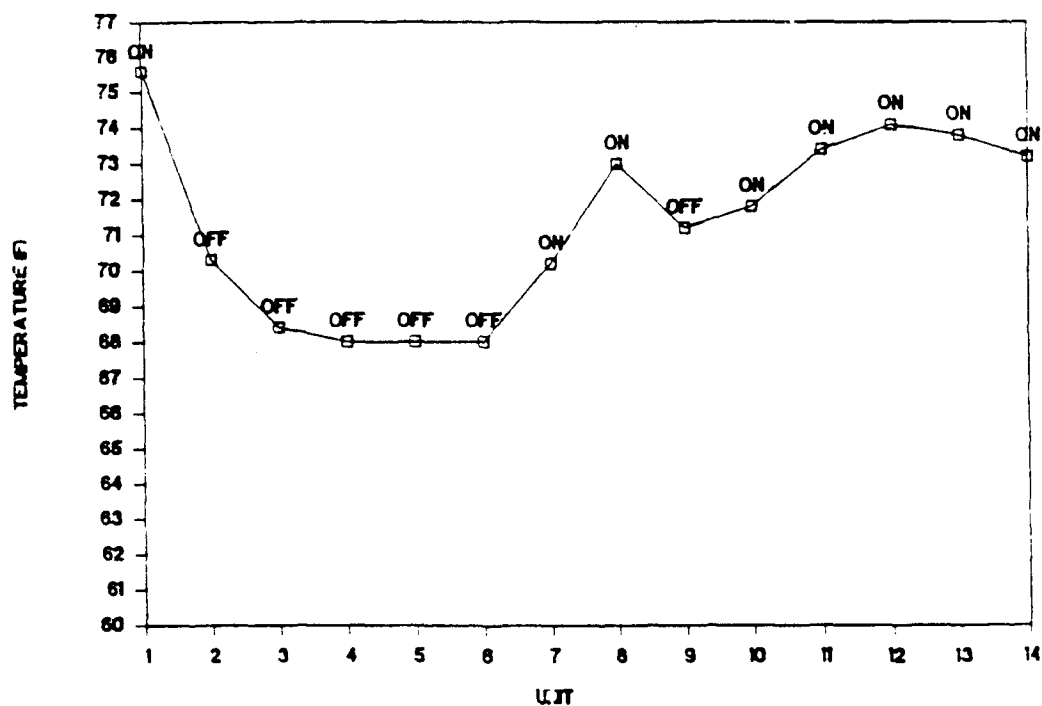


Figure 3. The effects of unit operation on the temperature recorded in the transport flume just downstream of the unit inflow (27 July 88). "ON" indicates an operating unit while "OFF" indicates non-operating units.

The district's assumption that the increase in mortalities observed at McNary Dam over the past few seasons is, in part, due to the thermal conditions of the collection system is supported by the data collected in 1987 and 1988. Rapid temperature changes in the collection system would have a synergistic effect on the level of stress normally associated with the collection system (Beil, 1986). Since previous studies (Maule, 1988) have determined that the collection process is often stressful and, given that the stress experienced appears to be cumulative, temperature variations through the collection system are probably a major contributing factor in the increased mortalities. However, the numerous changes in the operation of the powerhouse make it difficult to detect definite patterns between temperature fluctuations and changes in mortalities.

The stress associated with rapid temperature changes over space or time is often referred to as thermal shock. The rise or drop in temperature may be well within the fish's tolerant temperature range and yet still induce a significant level of thermal stress (Allen and Hassler, 1986). Rapid temperature changes have the effect of increasing the level of stress experienced by the juvenile fish (Strange et. al., 1977). Little guidance is provided in the literature on the magnitude or duration required for fish to pass through a temperature gradient without inducing thermal shock or aggravating their level of stress (Bell 1986).

With this information in mind, it appears that the two areas with the greatest potential for thermal shock are as the fish enter the transport flume from the gatewells and as they travel down the transport flume past other units. By identifying the areas that cause thermal stress, modifications may be made to try to reduce their impact. It appears that in other areas of the collection system, the fish have time either to avoid or adjust to the increased temperature.

One of the most likely areas to cause thermal stress is as the fish exit the orifice from the gatewell and plunge into the travel flume. The juveniles may come from a warm or cold water gatewell (depending on unit operation) and enter a cold or warm water flume (dependent on upstream unit operation). The temperature difference between the gatewell of an operating unit and the flume was as high as 6°F during the study period. This temperature change would be extremely rapid to any fish undergoing it.

The other area of concern relates to conditions as the fish are traveling down the flume to the holding pools. The fish may pass through a series of cold and warm water inflows from each orifice as they pass down the flume. The temperatures of these inflows are related to unit operation. Non-operating units pull collection water from the deeper, cooler water in the forebay while operating units draw in the warmer surface waters. For this study, the temperature rise or fall in the flume due to any particular unit was as high as 4°F. Data collected by the project personnel indicated that operating and non-operating units play an important role in the temperature of the flume near their orifices.

## CONCLUSIONS

An examination of all the available data collected during this 15-day period under study led to the following conclusions:

(a) Many of the assumptions relative to fish mortalities at the project made by the district and project personnel appear to be supported by the data. The exception is that the data do not support the theory that there is heating of the water by the dam structure or within the collection system.

(b) Powerhouse operations, specifically the start-up or shut-down of units, have a significant influence on the temperature seen in the collection system. Changes in total powerhouse discharge have only a slight influence on the temperature. The worst case for temperature fluctuations is when multiple units start up together or within a very short time (approximately 1 hour) of each other.

(c)  The operating units appear to be drawing in the warm surface waters from the forebay of the reservoir.  For reasons yet to be quantified, the units seem to draw the majority of their surface water from the southern shoreline.  As this water is brought in and drawn down into the penstock, it mixes with the cooler hypolimnetic water to form a zone of warm water in front of the operating units and to the south of the powerhouse.  Non-operating units draw their collection water from approximately the same elevation in the forebay as the entrance to the gatewell, hence their contributions to collection system are cooler than for operating units.

(d)  From the comparison of mortality versus powerhouse operation, the level of fish mortality seems to be closely related to changes in total powerhouse discharge and, in turn, coincides fairly well with units being brought on- and off-line (Wagner, 1987).  However, forebay surface water temperatures tend to increase the level of mortalities by inducing thermal stress in the juvenile fish.  The changes in temperature may magnify the normal stress associated with the collection system to a level above the lethal limit for some fish.  There may be other factors contributing to the mortalities, but numerous operational changes make these difficult to detect and identify.  However, in 1987 when this problem was first noticed, the powerhouse agreed to run at a flat-load, operating only the northern most units. This meant that no units would be brought on or taken off-line and that the total powerhouse discharge would remain relatively constant.  While the magnitude of mortality peaks decreased, the problem still persisted (Wagner, 1987).  This scenario may be the best estimate, to date, of mortality directly related to thermal stress since there seems to be very few other factors involved in the increased mortalities.

In an effort to minimize these problems, an attempt should be made to minimize the number, and/or magnitude, of the temperature gradients the fish must past through.  One recommendation is to close all orifices on non-operating units.  Correspondingly, the amount of water that is allowed to discharge through the ports on the side of the transport flume would have to be reduced.  From the observations of the fisheries personnel at the project, the number of fish collected by non-operating units is far less than for operating units so the overall efficiency of the collection system should not be seriously affected.[5]  By closing the orifices, the temperature of the water in the transport flume should be very similar to that in the gatewell and fairly uniform from one gatewell to the next.  This may have the greatest effect since the transition from gatewell to flume and travel down the flume results in almost instantaneous temperature changes over both space and time.

In addition, to reduce the mortalities and temperature fluctuations apparently related to the powerhouse discharge, starts of multiple units within a short period of time, or starting and operating one or more units separated by non-operating units should be avoided.  Multiple unit start-up can produce the greatest temperature changes in penstock and gatewell areas, while operating units separated by non-operating units can produce a significant temperature rise and fall in the water of the transport flume.

## ACKNOWLEDGMENT

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[5]Personal communication with Paul Wagner, Fisheries Biologist, Washington Department of Wildlife - McNary Dam, on 20 Apr 1989.

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# PREDICTING THE EFFECTS OF GENERATION, PUMPBACK, AND CHANNEL IMPROVEMENTS ON THE TEMPERATURE AND DISSOLVED OXYGEN OF J. STROM THURMOND LAKE

by

Thomas M. Cole[1]

## INTRODUCTION

This paper presents the results of numerical modeling studies determining the effects of proposed generation/pumpback operations and channel improvements on the temperatures and dissolved oxygen (DO) concentrations in the headwaters of J. Strom Thurmond Lake (JST).

The Savannah District has proposed adding four 75 MW pump/turbine units to the existing four conventional hydroelectric generation units located at Richard B. Russell (RBR) Dam. In addition, the District has proposed channel dredging in the tailwater region to improve flow conveyance. The Savannah District, State of Georgia, and Federal agencies are concerned that the proposed generation/pumpback operations and channel dredging may effect temperature and dissolved oxygen (DO) and alter fish habitat in the headwater region.

To address these concerns, the Savannah District requested a numerical modeling effort be conducted to determine the effects of generation/pumpback and channel improvements on temperature and DO in the JST tailwater region. CE-QUAL-W2, a two-dimensional, laterally averaged model was chosen to investigate the proposed changes.

## MODEL DESCRIPTION

CE-QUAL-W2 simulates variations in velocities, temperatures, and water quality in the longitudinal and vertical directions over time. It is capable of simulating a wide variety of water-bodies including rivers, lakes, reservoirs, and estuaries.

The model uses a finite differencing technique for the solution of the hydrodynamic and constituent transport equations. The water quality portion of the model consists of 20 constituents and includes nutrient/phytoplankton interactions. Other capabilities of the model include:

1.  Variable vertical grid spacing
2.  Automatic calculation of the timestep necessary to maintain numerical stability
3.  Multiple branches
4.  Restart capabilities which allow simulations to be stopped at any time and then restarted later
5.  Multiple inflows and outflows including branch inflows, point and nonpoint sources, precipitation, outlets, and withdrawals

## STUDY AREA

The study area included only the upper 10 km of JST Lake because preliminary evaluations indicated that the impacts of generation, pumpback, and channel improvements on JST Lake would be most evident in the headwater region. A map of JST Lake showing the study area and sampling stations is shown in Figure 1.

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[1]Hydrologist, Water Quality Modeling Group, U.S. Army Engineers Waterways Experiment Station
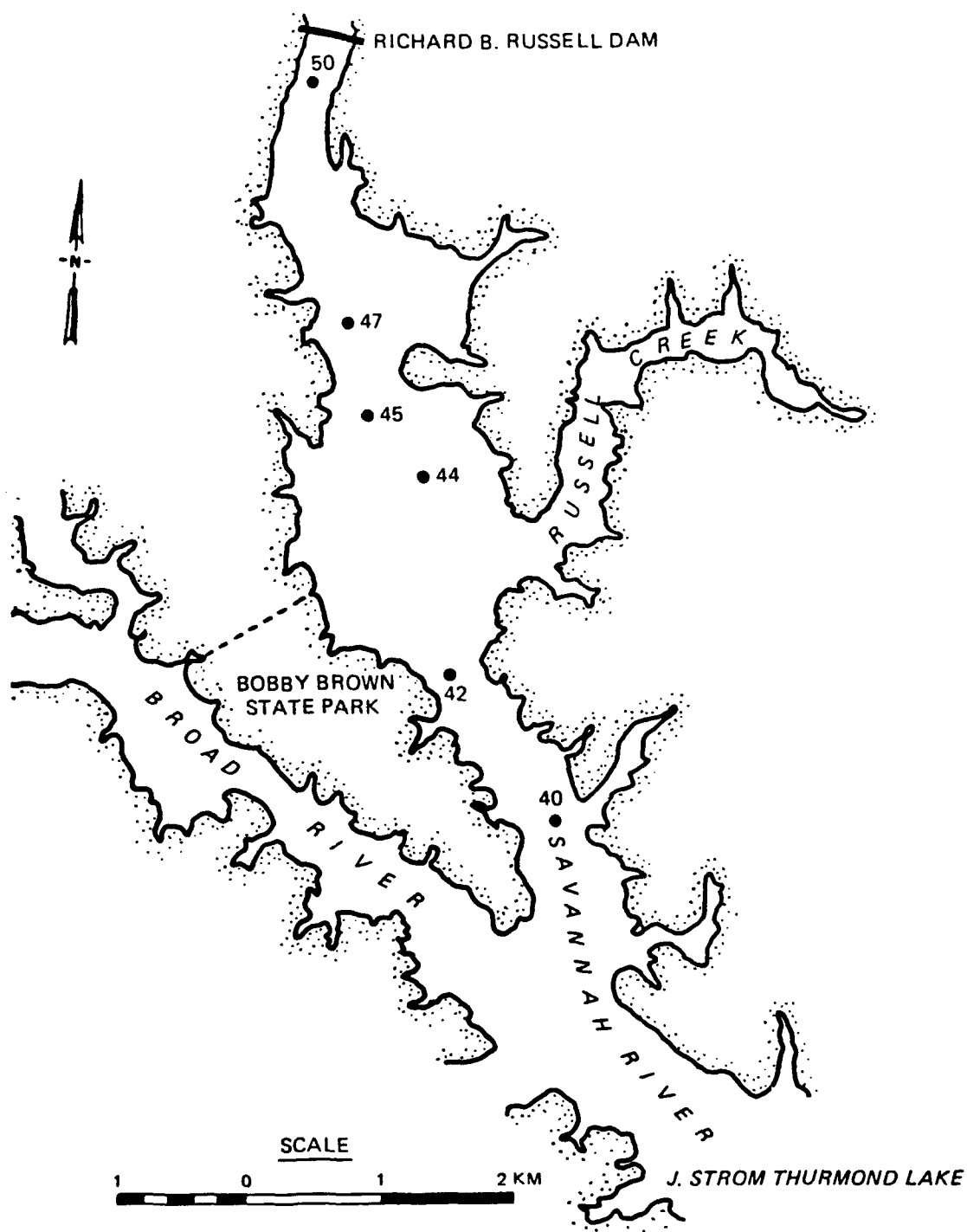
Figure 1. Map of study area showing sampling station locations.

The RBR Dam includes four 75-MW, conventional hydroelectric units and room for four proposed 75-MW pump/turbine units. For generation, each unit has a rated capacity of 7,500 cfs; each giving a total generation capacity of 60,000 cfs. For pumpback, each unit has a pumping capacity of 6,200 cfs giving a total pumping capacity of 24,800 cfs.

## CALIBRATION AND VERIFICATION

One of the most important tasks in a successful application of any numerical model is obtaining good calibration and verification data. The two sets should be representative of as widely varying conditions as possible. This application used data from two different years, pool elevations, and RBR Dam operating schedules. Calibration data were collected during August 13-15, 1984, when the pool elevation was close to conservation pool and releases at RBR Dam were through the tainter gates. Verification data were collected during June 22-25, 1986, when the pool elevation was about two meters lower than during the 1984 period and releases at RBR Dam were through the turbines.

Calibration results for temperature and DO are given in Figure 2 with station locations given in Figure 1. The initial conditions for the simulation are also given. Temperature predictions are generally in agreement with observed data except that the model misses the placement of the thermocline at station 45 by roughly one meter. The model correctly captured the $2°C$ temperature increase in the hypolimnion and the $2-10°C$ cooling of the epi- and metalimnion. DO predictions exceed measured values by approximately 1 mg/l in the hypolimnion at station 40.

Verification results are given in Figure 3. Temperature predictions are again generally in agreement with observed data except for a slight misplacement of the thermocline at station 45. Note that the model captured the roughly $14°C$ temperature decrease in the hypolimnion over the three-day period. DO predictions are generally in agreement with observed data.

Several points should be kept in mind when evaluating the accuracy of the predictions. First, results from the model represent a value averaged over 500 m in the horizontal and 0.5 m in the vertical over a timestep (100-250 sec), whereas observed data are taken at discrete points in space and time. Also, the predicted values for all stations were output from the model at the same point in time, whereas observed data differed by as much as 45 minutes from the upstream and downstream stations. Also, all models suffer from limitations imposed by their basic assumptions and the solution scheme employed. The major assumptions in CE-QUAL-W2 important to this study are that lateral variations in flow and water quality are negligible, as are vertical accelerations. In addition, the model uses an upwind differencing scheme which introduces numerical diffusion into the results which can be greater than the physical diffusion.

## SIMULATIONS

Simulations were conducted for the following eight scenarios (proposed changes are shown in **bold** type):

high pool
- generation/no pumpback — no channel improvement / **channel improvement**
- generation/**pumpback** — no channel improvement / **channel improvement**

medium pool
- generation/no pumpback — no channel improvement / **channel improvement**
- generation/**pumpback** — no channel improvement / **channel improvement**
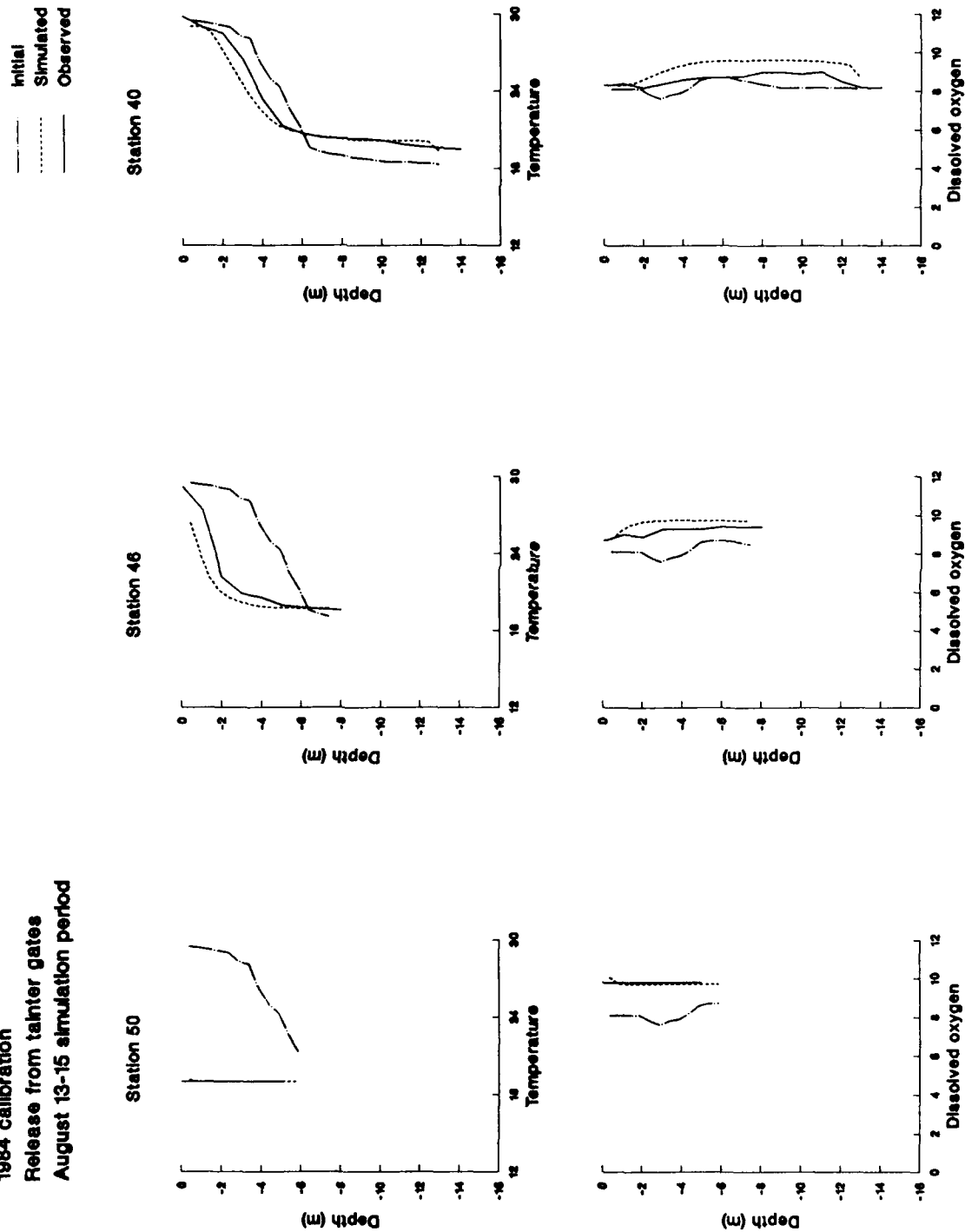
Figure 2. Results of calibration run for temperature and DO.

100
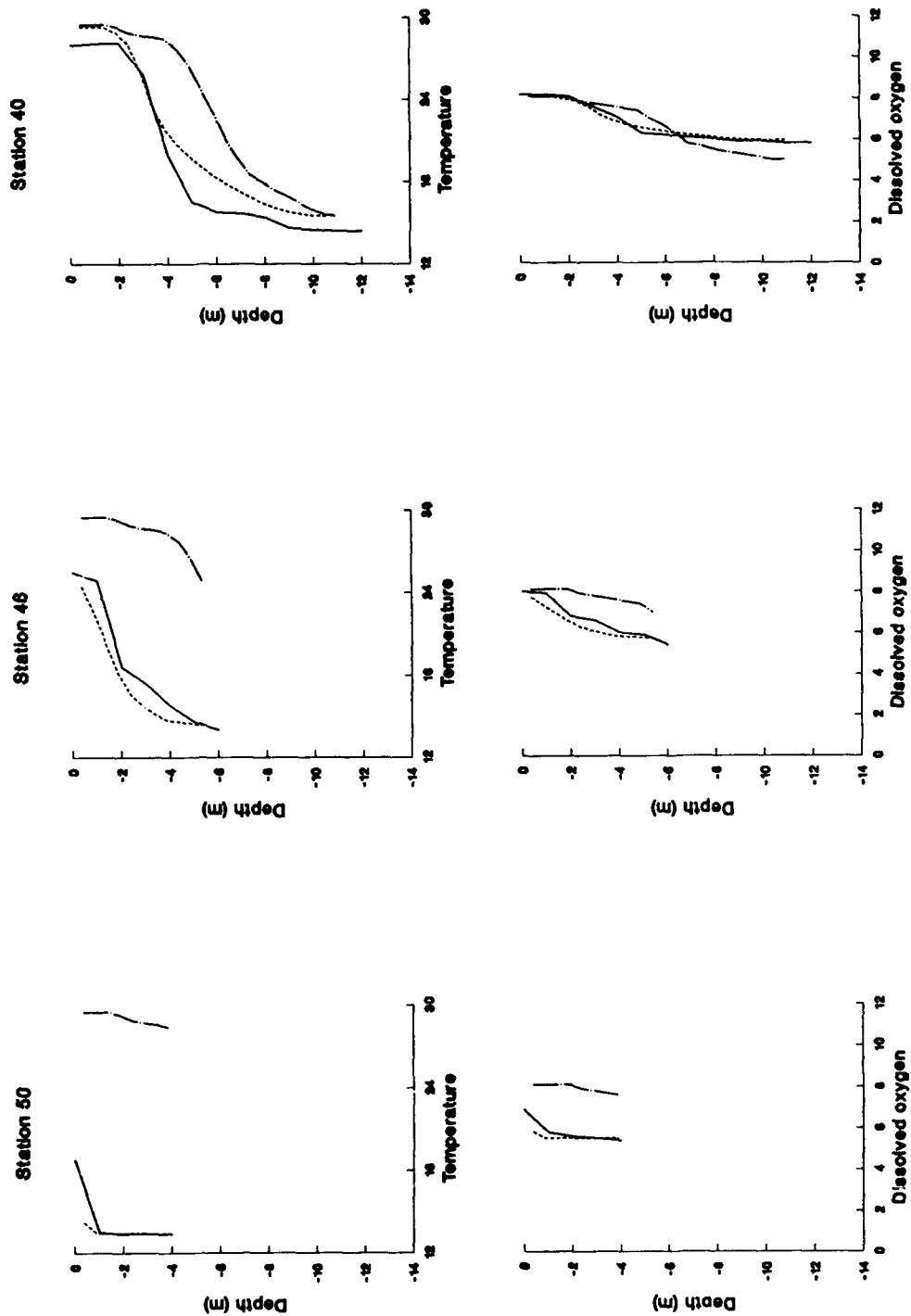
Figure 3. Results of verification run for temperature and DO.

High pool was at conservation pool (el. 330 ft.) and medium pool was at 323.5 ft. An additional set of simulations was planned for elevation 316.9 ft., but the generation cycle produced supercritical velocities which the model was not designed to handle. Peak generation was set at 60,000 cfs and pumpback was set at 20,000 cfs. The proposed generation/pumpback schedule is shown in Figure 4. No generation/pumpback was proposed for weekends.



Figure 4. Proposed summer generation/pumpback schedule for weekdays.

The proposed channel improvements include excavating to an elevation of 317 ft. a 300 m wide channel that extends for approximately 1.7 km downstream of the coffer-dam. They are shown in Figure 5.

A search of the historical records showed that the greatest degree of stratification and lowest DO values occurred during August 1987. Consequently, the model was initialized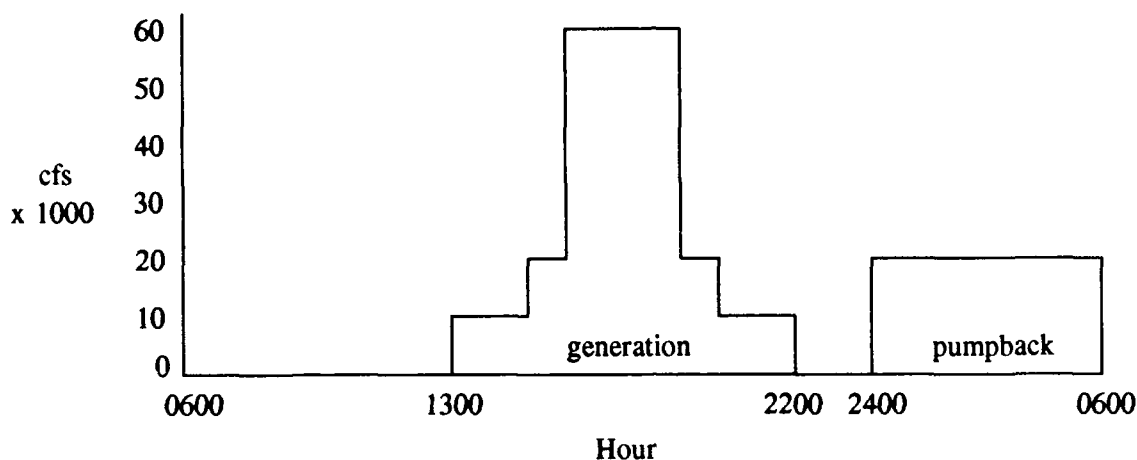 to these starting conditions, in keeping with a "worst case scenario" approach. Inflows and outflows for generation and pumpback were set to the values for the proposed summer operating schedule (Figure 4). Inflow temperature from RBR Dam was set to the highest observed temperature and DO inflow concentration was set to the mandated minimum release concentration of 6 mg/l. The duration of all simulations was one week.

## RESULTS

Figure 6 shows the results of the simulations at medium pool for generation with no pumpback or channel dredging. The effects of the proposed changes at high pool elevation were not as pronounced and are not presented in this paper. Power generation caused a decrease in temperature and an increase in DO the length of the study area. The effects of power generation on temperature were most pronounced near the dam (segment 2) and decreased downstream until they were barely noticeable. Changes in DO were most pronounced at the most downstream segment (segment 20).

Figure 7 shows the results of the simulations at medium pool for generation with pumpback and channel improvements. The same trends in temperature and DO changes were also evident for this simulation. However, the changes in temperature were not as pronounced as the simulation involving generation only. The increases in hypolimnetic DO were more pronounced near the dam and less pronounced at the downstream segment.

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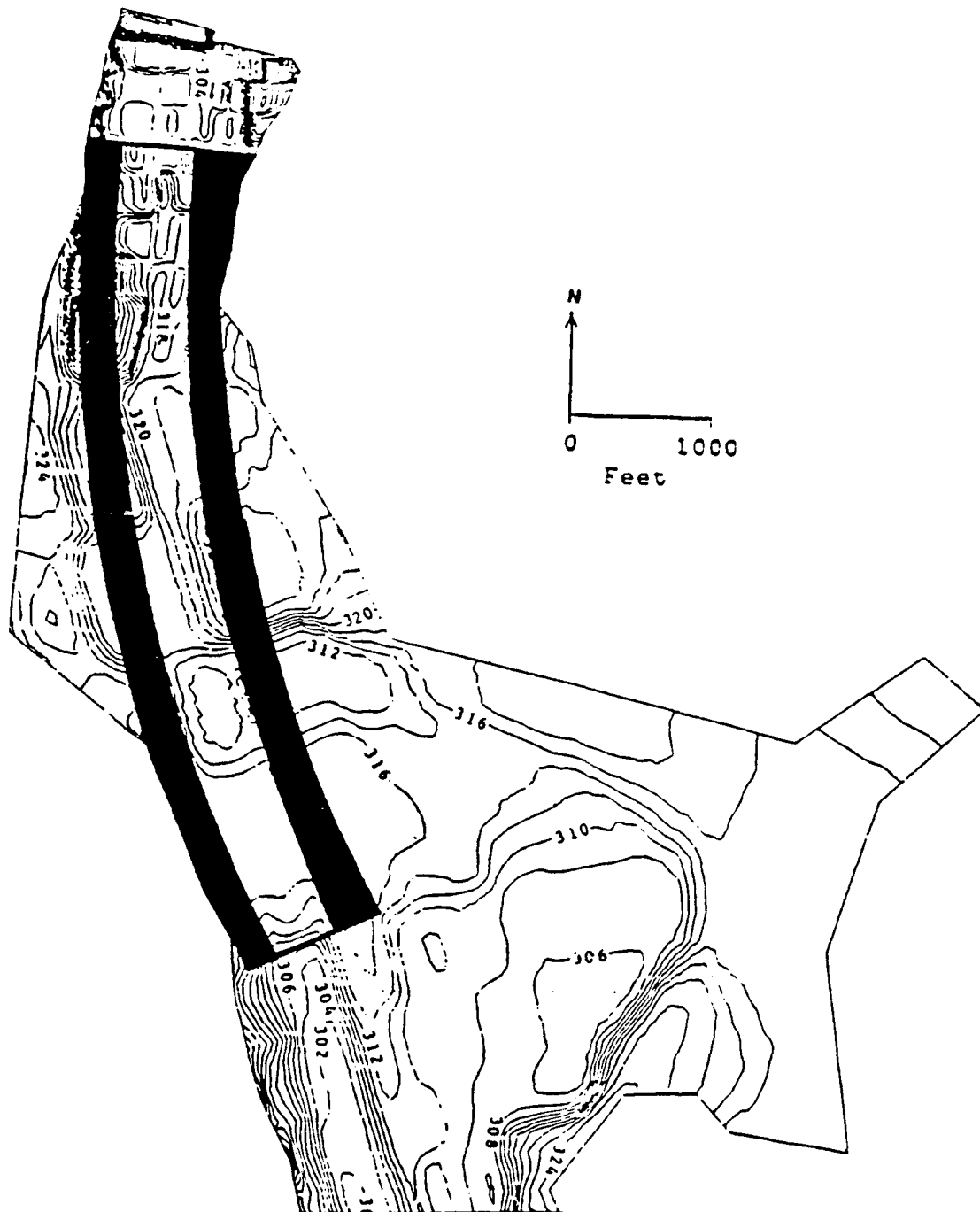
Figure 5. Proposed location of channel excavation.

Figure 7. Temperature and DO results for generation/pumpback and channel improvements at medium pool.

J. Strom Thurmond Lake
Proposed summer power generation schedule
No pumpback or channel improvement
Medium pool elevation
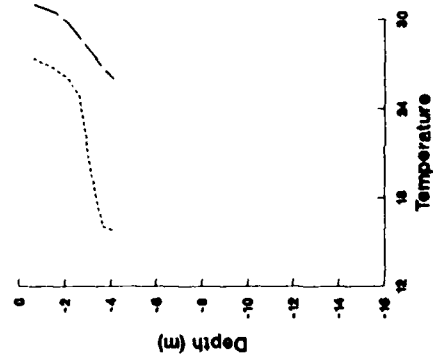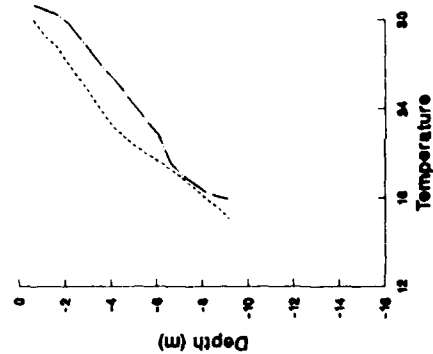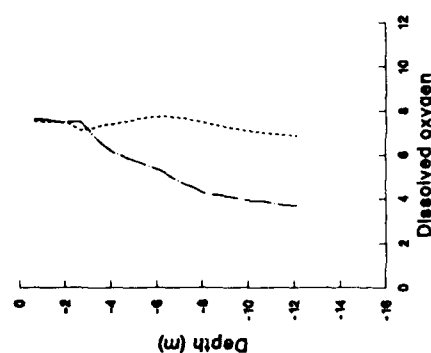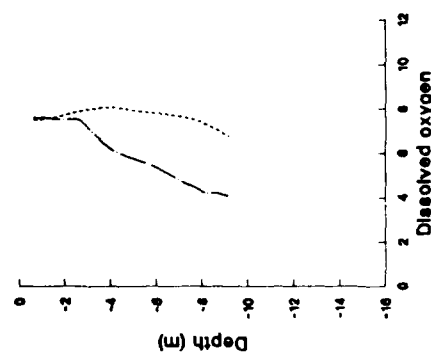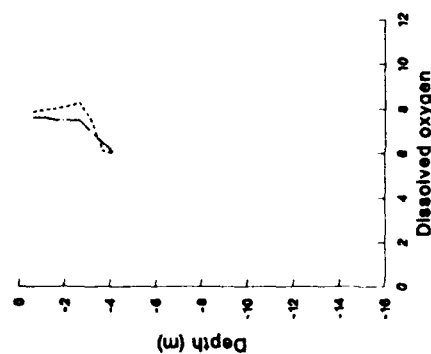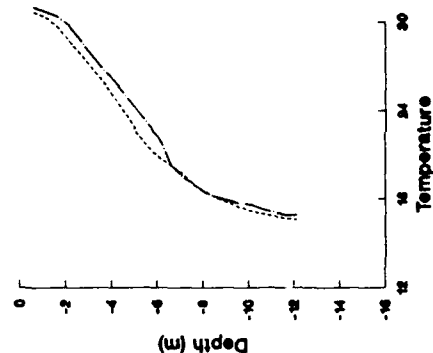
Figure 6.  Temperature and DO results for generation only at medium pool.

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## CONCLUSIONS

CE-QUAL-W2 proved to be an effective tool to investigate the proposed generation/pumpback and channel improvements in J. Strom Thurmond Lake. Results showed that the proposed changes would decrease temperatures and increase DO concentrations in the hypolimnion thus having a minimal detrimental impact on the cold-water fish habitat in the tailwaters. The simulations helped address concerns expressed by the Savannah District, State of Georgia, and Federal agencies regarding the proposed changes to Richard B. Russell Dam and J. Strom Thurmond tailwaters.

## ACKNOWLEDGEMENTS

# APPLICATION OF ASPIRATING HYDRAULIC MIXER TO IMPROVE TAILWATER DISSOLVED OXYGEN

by

Richard E. Price[1]

## INTRODUCTION

A number of Corps of Engineers (CE) Reservoir Projects experience low dissolved oxygen (DO) levels in the tailwater during the stratified periods cf the year. In some cases, the loss of DO results in fish kills in the stilling basin or downstream. These have been reported both during release and non-release periods. In recent years, a number of fish kills have been reported in the stilling basin and downstream from Lake Texoma. During the stratified period, which usually begins in May of each year, hypolimnetic oxygen demand in the reservoir begins to deplete the dissolved oxygen (DO). By July, portions of the hypolimnion become anoxic. Since the intake structure is located near the bottom, release of low DO water occurs. This continues until the lake stratification breaks up in the fall and reaeration of the hypolimnion occurs.

At the request of Tulsa District (SWT), Punnett and Culbertson (1981) investigated the cause of the fish kills and concluded they were the result of release of low DO water. To alleviate this condition, they recommended small releases (50 cfs discharge termed "fish releases") through the flood control structure to provide turbulence for reaeration downstream. Fish kills have continued since the initiation of this procedure in 1981; however, the severity has declined.

A similar situation exists at Walter F. George Lock and Dam on the Chattahoochee River in Mobile District. During the late summer when conditions are conducive for release of low DO water, relatively small releases are made through the spillway gates during nongeneration periods to reaerate the tailwater and prevent fish kills (Findley and Day, 1987).

Recently, this low-release operation has been recognized as a source of scour on the gate sill at Texoma. In addition, this release of water reduces the available head for generation. Therefore, alternatives to this operation are needed to provide adequate DO in the stilling basin during the stratified period without impacting the structure.

As part of the Water Quality Research Program work unit entitled "Hydraulic and Pneumatic Mixers and Aerators in Principle and Practice", a literature review indicated that an aspirating surface mixer which entrains air in a hydraulic jet may be ideally suited for low flow situations. These types of aerators are particularly suited to relatively shallow areas which are not strongly stratified. The combination of air bubbles in a hydraulic jet improves the reaeration efficiency of the device. Although this type of aerator may solve the DO problem, field experience with these types of devices at CE projects is limited. A pilot study to investigate the efficiency of reaeration of this type of device as well as the feasibility of its use at CE projects was conducted in the tailrace of the hydropower facility at Lake Texoma during the summer of 1989. Results of this field study were used to develop evaluation techniques for tailwater aeration devices.

## Project Description

Lake Texoma and Denison Dam, operated by the Tulsa District (SWT), is a flood control and hydropower project located on the Red River (mile 725.9) on the state line between Oklahoma and

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Texas, approximately 5 miles northwest of Denison, Texas. This 5.27 million acre-feet reservoir has a surface area of approximately 143,000 acres at maximum storage. At the normal power pool volume, the storage capacity is approximately 1.67 million acre-feet with a surface area of 89,000 acres and a maximum depth of approximately 96 feet. The hydropower project is operated as a peaking project with two units generating up to 70,000 kw at a hydraulic discharge capacity of approximately 10,500 cfs.

The flood control outlet works for this project consists of three 20-foot diameter concrete conduits through the embankment controlled by six 9 x 19 foot vertical lift gates. The two 35,000 kw hydropower units are controlled by four 9 x 19 foot vertical lift gates with the sill located at elevation 523.0 in the intake structure. The reservoir bottom is near elevation 521.0 in the vicinity of the intake structure. During nongeneration periods, the project is operated from power generated by one of two house units, which discharge approximately 25 cfs per unit.

## DENISON DAM TAILWATER DO

The need for reaeration of releases from Lake Texoma during the late summer is the result of release of hypolimnetic water which is low in DO. Even though the thermal stratification is weak during the late summer with a surface-to-bottom temperature difference of only 6° to 8° C, the DO stratification is usually significant, with the surface at or near saturation and the bottom anaerobic.

Reservoir release operations during this period center on hydropower generation. Once the normal power pool elevation is reached after spring flood releases, peaking hydropower releases, which are made on a day-by-day basis, are the only scheduled releases. The nongeneration releases from the hydropower facility are composed of discharge from the house unit and leakage from the wicket gates of the two main units. When generation is terminated, the turbines are unloaded and water is allowed to pass through the turbines to cool the system down prior to complete cessation of flow through the turbines. As the wicket gates are closed, air is drawn in through the vacuum breaker system (normally closed during generation) to the region downstream of the turbine and entrained into the flow. This causes considerable reaeration of the tailrace as indicated by the bubbles observed downstream during this process.

During the remainder of the nongeneration period, the gate leakage continues to pass water into the tailrace. The leakage flow through the penstocks comes almost exclusively from the hypolimnion and reflects the low DO present there. Therefore, the DO in the stilling basin will begin to decline as a result of inflow of low DO water via gate leakage and, to a lesser extent, oxygen depletion as a result of the hypolimnetic oxygen demanding materials.

During the reaeration tests conducted in August 1989, the DO in the stilling basin was monitored during generation and nongeneration periods to establish the extent of reaeration needs during the generation and nongeneration periods. Figure 1 is a plot of DO in the tailrace and house unit discharge for August 27 and 28. Hydropower generation ceased at 20:00 hours and did not begin again until 12:00 hours the following day. During generation, the release DO was approximately 4.0 mg/l. Upon the initiation of the shut down sequence, the tailrace approached saturation with respect to DO due to aspiration of air through the vacuum breaker system. Once all flow was stopped, the DO began to decline and reached equilibrium at 0.2 mg/l approximately 12 hours later.

The discharge DO from the house unit followed a different sequence. Once the main units ceased operation and discharge, the tailwater began to drop creating a low pressure region downstream of the house unit turbine. The vacuum breaker system then opened and allowed air to be entrained into the flow in the draft tube. This provided some reaeration of flows and maintained the DO concentration in the release from the house unit (at approximately 3.1 mg/l during the tests in August 1989) even though this water came from the hypolimnion in the reservoir (Figure 1).
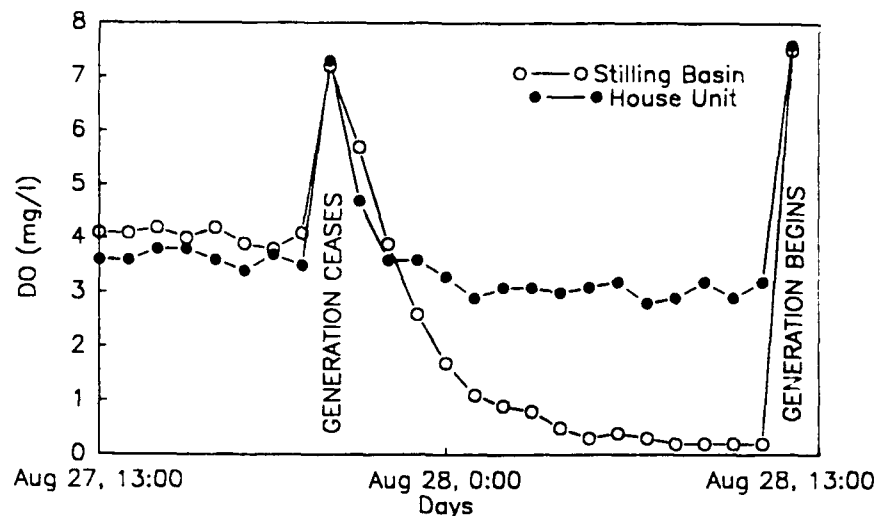
Figure 1.  Hydropower Tailrace Dissolved Oxygen for August 27 and 28, 1989.

## ASPIRATING AERATOR TESTS

Aspirating aerators are particularly suited to relatively shallow areas which are not strongly stratified, such as those found in stilling basins (Price, 1988).  The combination of air bubbles in a hydraulic jet improves the reaeration efficiency of the device.  The simplicity of the aerator design in that there are no compressors, lines or diffusers as with a pneumatic aeration system minimizes operational and maintenance costs.  In addition, the ability to quickly remove the device from the tailrace for initiation of hydropower generation is a necessity.

The reaeration tests conducted in the tailrace were designed similar to those used to test aeration devices for oxygen transfer.  The tailrace was considered to be a mixing basin with inflow from the draft tubes and outflow over the end sill downstream.  Basin volume was determined from project drawings as 124,500 cu. ft. (.931 mil. gallons).  Flow-through rate (leakage rate through the main turbines) was estimated at 15 cfs based on leakage rates from similar projects.  Based on this volume and flow-through rate, a 15-Hp aerator should provide adequate concentrations of DO leaving the tailrace.   For example, to input 5 mg/l DO into a flow of 15 cfs would require 16.8 lb/hour of oxygen.  At a rate of 2.5 lbs $O_2$/Hp/hour$^2$, which is the reported aeration rate for an aspirating aerator, this would require 6.7 Hp.  Therefore, to provide a range of oxygen input rates for evaluation of aeration efficiency, 3-, 5- and 10-Hp aerators were leased for testing in the tailrace.

### Field Tests of Aerators

Since the low DO in the tailrace usually begins in the late summer, testing of the mixers began August 27, 1989 and were completed 1 September.  Set-up of monitoring equipment and measurement of the tailrace began on 27 August.  Hydropower generation stopped at 20:00 hours on 26 August so the only flow in the tailrace was from gate leakage and the house unit.  Measurements of the tailrace revealed that the end sill was 120 feet from the draft tube with a width of 120 feet and the depth at the draft tube exit was 30 feet, decreasing to the tailrace end sill with a depth of approximately 1 foot.  This increased the volume of water in the tailrace to 216,000 cu. ft. (1.600 mil gallons).

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[2]Provided by Aeration Industries, Inc., Chaska, MN  55318.  Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products.  The contents of this report are not to be used for advertising, publication, or promotional purposes.

The flow rate was measured at 10 foot intervals over the end sill, at the surface, mid-depth and near the bottom. Beginning at the north end of the sill, measurements indicated a velocity of approximately 0.2 ft/sec, increasing to 1.1 ft/sec near the center of the sill and declining to 0.4 ft/sec at the south end. With a depth of approximately 1 foot, a discharge of 76 cfs was computed from these measurements. Velocity from the house unit was also measured at a consistent 0.95 ft/sec. Since this conduit was rectangular (5 ft by 8 ft) with approximately 5 feet submerged, the discharge was computed as 25 cfs. Since the total flow leaving the tailrace was 76 cfs, with 25 cfs from the house unit, the gate leakage, which was the remaining flow, was approximately 51 cfs.

With the basin dimensions and flow rate established, the monitoring of DO in the tailrace during generation and nongeneration began. Operation of the hydropower facility during these tests consisted of daily generation from 12:00 to 20:00 hours. Monitoring of the DO in the tailrace during the generation period indicated that the release DO was relatively constant at 4.0 mg/l during these tests. Once the generation shut-down procedure was initiated, the release became saturated with DO. The DO then declined until a steady state condition of 0.25 mg/l DO was reached (Figure 1). The mixing of the house unit discharge with the gate leakage resulted in the DO leaving the tailrace of approximately 1.6 mg/l. The evaluation of aerator efficiency, which will be discussed later, requires the determination of the oxygen depletion rate in the tailrace. This can be determined by monitoring the DO in the tailrace with time. The largest rate of decline in DO was approximately 1.8 mg/l/hr. However, this is a reflection of the low DO water diluting and eventually filling the tailrace rather than a rate of oxygen depletion. If this were a true oxygen depletion rate, it would equate to a BOD of approximately 216 mg/l, a value close to that of domestic sewage. Punnett and Culbertson reported the BOD ranging from 1 to 4 mg/l in the release at Lake Texoma. This would equate to a DO depletion rate of 0.01 to 0.03 mg/l/hr, which is more representative of hypolimnetic oxygen depletion rates.

The tests of the aerators began, with the 5-Hp unit being lowered into the tailrace. This unit was positioned in the center of the tailrace, approximately 10 feet from the draft tube wall. It was positioned such that the hydraulic jet issued toward the south wall, perpendicular to the flow from the hydropower draft tube. The vacuum breaker on the house unit was closed when hydropower generation ceased at 20:00. The aerator was turned on at 24:00. By 02:00, the DO was 0.6 mg/l, which is 0.35 mg/l higher than the 0.25 mg/l reached after six hours of nongeneration on the previous day. At this point, the vacuum breaker on the house unit was opened and the turbine allowed to aspirate. This increased the DO leaving the tailrace to 2.18 mg/l. The aerator was rotated 180 degrees to face the jet toward the north end of the tailrace. This resulted in a decrease in the DO to 1.84 mg/l. To verify that the mixer was increasing the DO, the aerator was turned off to determine if the DO would decline. When it was turned off, the DO dropped to 1.65 mg/l. Therefore the 5-Hp unit increased the DO concentration leaving the tailrace by approximately 0.2 mg/l while the contribution from venting of the house unit was approximately 1.6 mg/l.

The next tests were conducted using a 10-Hp and a 5-Hp unit. Since the 0.2 mg/l increase observed in the previous test was relatively small, the 10- and 5-Hp units were operated together to simulate a 15-Hp unit. With the house unit vacuum breaker closed and the two aerators operating, the DO leaving the tailrace declined to 1.4 mg/l. This was an increase of 0.5 mg/l from the test using only the 5-Hp aerator and a 1.15 mg/l increase with no aerator and the vacuum breaker closed. Once the vacuum breaker was opened, the DO increased to 2.4 mg/l. If the house unit vacuum breaker system was contributing 1.6 mg/l as with the 5-Hp tests, the 15-Hp aeration capacity contributed 0.8 mg/l. To determine if the aspiration ability contributed to the reaeration, the air intakes for the aerators were taped shut to prevent aspiration. The DO began to decline and after 2 hours, the release DO had declined to 1.79 mg/l. Therefore, the decline of approximately 0.6 mg/l indicates the aspiration ability is important in the reaeration process. When the aerators were turned off, the DO dropped to 1.8 mg/l, indicating the aerators were contributing 0.6 mg/l DO. Thus the 15-Hp aeration capacity contributed from 0.6 to 0.8 mg/l DO to the tailrace.

To summarize the results of these tests, the contribution to the DO in the flow leaving the tailrace from each source was identified. The gate leakage flow of approximately 51 cfs maintained a DO of approximately 0.25 mg/l. The house unit with its 25 cfs discharge also had the same DO when the vacuum breaker was closed. When it was opened, DO increased to 3.1 mg/l. The DO leaving the tailrace with the house unit vacuum breaker open with 25 cfs containing 3.1 mg/l and 51 cfs containing 0.25 mg/l was 1.6 mg/l. The 5-Hp aerator increased the DO approximately 0.2 mg/l and the 5- and 10-Hp combination increased the DO approximately 0.6 mg/l (Figure 2).
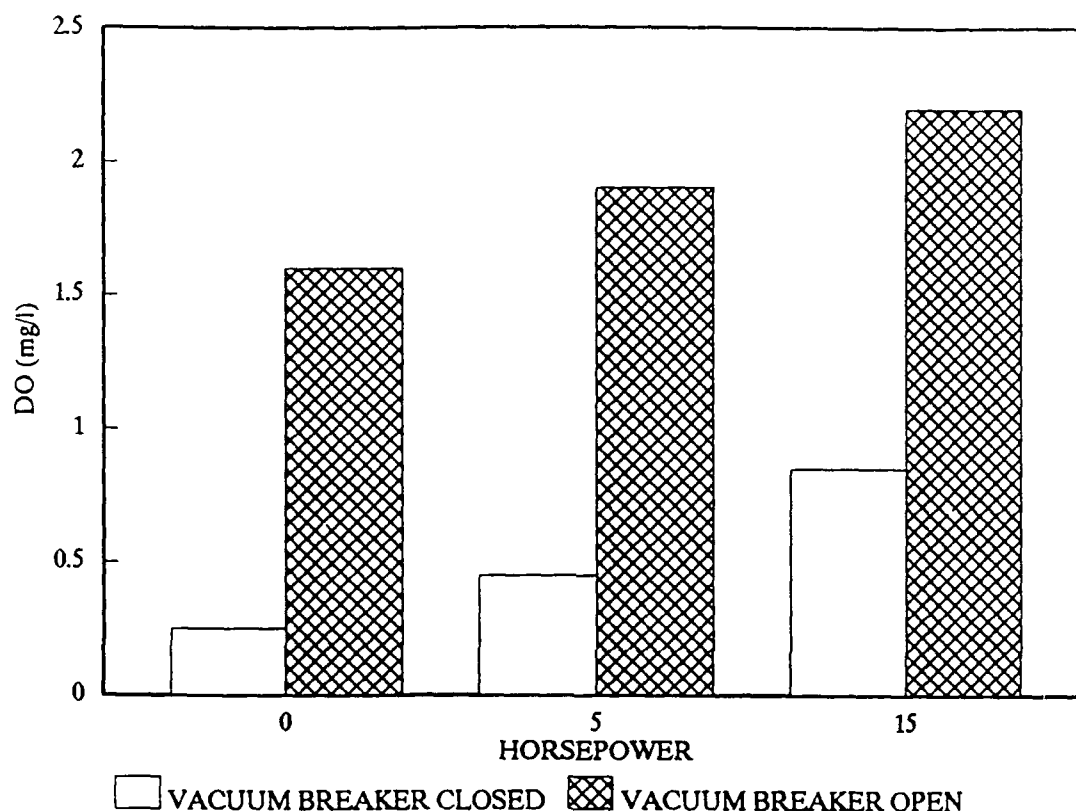


Figure 2. Tailwater Dissolved Oxygen improvement with aspirating aerators.

Evaluation of Aspirating Aerator

The objectives of these tests were to evaluate the DO dynamics in the hydropower tailrace during nongeneration periods, and to evaluate the feasibility of reaerating the gate leakage flows with an aspirating type aerator. The monitoring of DO in the tailrace indicated that the amount of reaeration that occurred through the house unit resulted in a DO of approximately 1.6 mg/l leaving the tailrace. The tests of the aerators indicated increased DO leaving the tailrace of 0.2 to 0.6 mg/l with the 5- and 15-Hp tests, respectively. Since the volume of the tailrace and nongeneration flow rate for the tailrace were underestimated, the improvements to the release DO were minimal. With a tailrace flow rate of 76 cfs, DO input to achieve 5 mg/l at this flow rate would be 83.8 lbs of $O_2$/hr. This would require a 35-Hp unit at 2.5 lbs of $O_2$/Hp/hr. Since the tailrace is much larger than the drawings indicate, the required Hp to achieve a given release DO would be larger.

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The dynamics of the DO in the tailrace were described above, with the DO increasing from approximately 4.0 mg/l to saturation with the unloading of the turbines during the shut-down sequence of the main turbines. The tailrace was then gradually filled with low DO water from the gate leakage, a process that occurred in about 6 hours from the cessation of generation. As indicated previously, the aerators were leased on the basis of a gate leakage of approximately 15 cfs and a smaller tailrace volume. With a leakage of 51 cfs and the larger size of the tailrace, the aerators were only capable of aerating a portion of the release flow. One objective of these tests was to identify the key factors affecting the DO in the tailrace so that a numerical model could be developed/adapted to evaluate aerators in tailrace environments. From this, criteria for design of tailrace aeration systems could be given.

Considerable research has been conducted on oxygen transfer based on the two-film model (ASCE, 1983). This model is the recommended model for analysis of unsteady state oxygen transfer test data and is based on a transfer coefficient multiplied by the driving gradient for a given aeration system and test basin.

$$\frac{dC}{dt} = K_L a(C_\infty^* - C) \tag{1}$$

where:

$C_\infty^*$ = average dissolved oxygen saturation concentration (mg/l)
$C$ = dissolved oxygen concentration at time, $t$
$K_L a$ = apparent volumetric mass transfer coefficient, $t_{-1}$
$t$ = time

This model can be modified to estimate the oxygen transfer coefficient for the tailrace tests assuming the tests were steady state continuous tests. The driving force for reaeration is affected by the flow-through rate as well as the volume of water contained in the basin. These factors influence the ability of the aeration device to fully mix the basin. To account for the flow-through and the volume of the tailrace, the model is modified to the following form:

$$K_L a = (R - (Q/V)(C_o - C_R))/(C_\infty^* - C_R) \tag{2}$$

where:

$R$ = oxygen uptake rate (mg/l/hr)
$Q$ = hydraulic flow through rate (mil gal/hr)
$V$ = tailrace volume (mil gal)
$C_o$ = inflow DO concentration (mg/l)
$C_{R_*}$ = tailrace DO concentration (mg/l)
$C_\infty^*$ = tailrace DO saturation concentration (mg/l)

Once the oxygen transfer coefficient has been determined, the aerator efficiency can be determined using the following equation:

$$OTR = (K_L a * 8.34 * V * C_\infty^*)/H \tag{3}$$

where:

$OTR$ = oxygen transfer rate (lbs $O_2$/Hp/hr)
$H$ = horsepower of aerator

For the tailrace conditions during these tests, the DO saturation concentration ($C_\infty^*$) was 8.74 mg/l, flow rate (Q) was 76 cfs (2.04 mil gal/hr) and the tailrace volume (V) was 1.61 mil. gal. Based on the discussion above, oxygen uptake rates (R) of .001 to .01 mg/l/hr were used. From equation (2), the oxygen transfer coefficient ($K_La$) for the 5-Hp test was approximately 0.035. When converted to aeration efficiency using equation (3), this equates to 1.8 to 1.9 lbs $O_2$/Hp/hr. With the 15-Hp test, the oxygen transfer coefficient was 0.121 with an aeration efficiency of 2.1 to 2.2 lbs $O_2$/Hp/hr.

Although this provides a means of evaluating the efficiency of the aerators, it does not provide a means to determine the required Hp to achieve a given release DO at Lake Texoma. To do this, the increase in tailrace DO with the 5- and 15-Hp tests must be viewed in light of the DO deficit satisfied by each aerator. With the DO deficit at 7.14 mg/l, the 5-Hp unit satisfied 0.2 mg/l (2.8 % of the deficit) and the 15-Hp satisfied 0.6 mg/l (8.4 % of the deficit). When divided by the number of Hp, the units satisfy 0.56 %/Hp. Therefore, if a release DO of 3.5 mg/l is desired (a DO increase of 1.9 mg/l or a reduction of 27 % of the deficit), then 50-Hp of aeration capacity is required.

The relationship between aerator Hp and desired release DO is shown in Figure 3, along with the observed release DO concentrations for the 5-Hp and 15-Hp aerator tests. As the DO in the tailrace approaches saturation, the amount of energy required to achieve an increase in DO increases exponentially. If the leakage rate increases or the oxygen depletion rate increases significantly, then the required Hp to achieve a given release DO would also increase. Since this relationship was developed from the tests conducted in the tailrace at Lake Texoma, it is only suitable at this project. Other projects exhibiting similar problems would most likely have different tailrace dimensions, leakage rates, and oxygen depletion rates and would require testing prior to implementation of a tailrace aeration system.
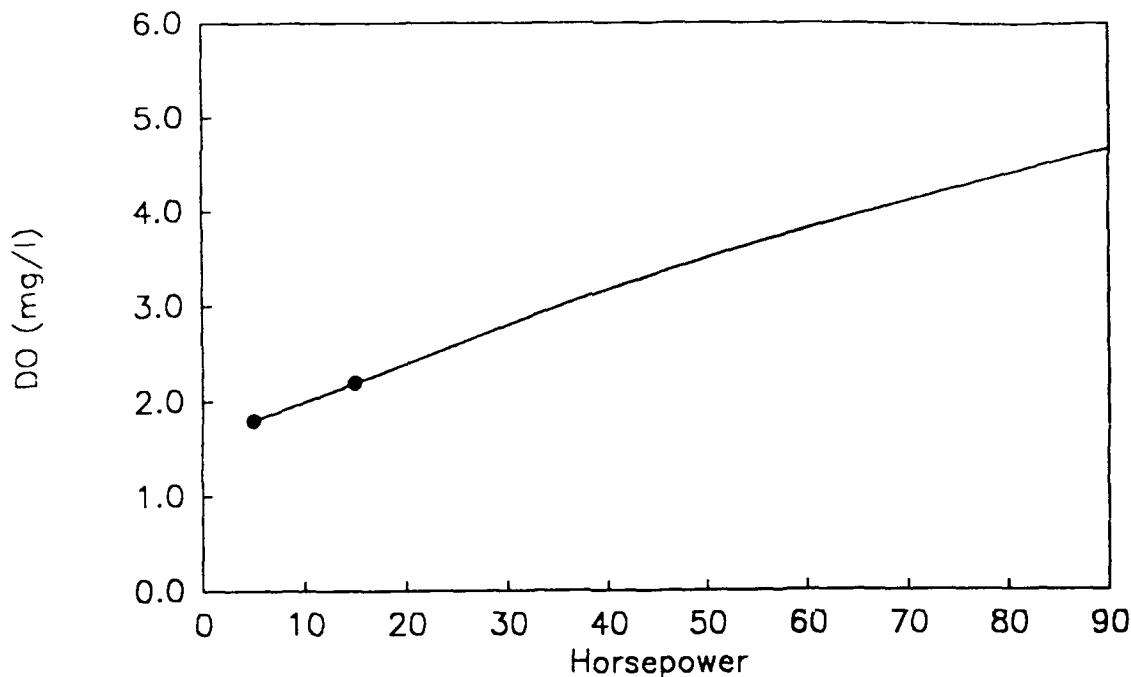


Figure 3. Aerator Horsepower for desired release dissolved oxygen concentration from Denison Dam.

# CONCLUSIONS

The tests of the aspirating aerators in the stilling basin indicated that these types of aerators can be used to increase nongeneration release DO. Evaluation of the aerators indicated lower efficiencies than those reported by the manufacturer. Since the amount of reaeration is based on the Hp of the aerator, a curve was developed to define the Hp required to achieve a given release DO in the Denison Dam tailrace. At a minimum, a 50-Hp unit would provide a release DO of 3.5 mg/l. A single unit could be mounted on a track system to allow for automated operation and retrieval from the tailrace during generation periods.

# ACKNOWLEDGMENT

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# TEMPERATURE CONTROL IMPACTS WITH WHITEWATER RELEASES
## AT SAVAGE RIVER DAM PROJECT

by

Kenneth S. Lee[1]

## INTRODUCTION

The Savage River below Savage River Dam is a well known place for whitewater sports. Whitewater releases from the project have been made since the early 1970's. The releases were generally made in late August or early September. The magnitude of the releases was about 800-1000 CFS and the length of the release was for three days (about 2,000 acre-feet of storage). Whitewater sports are growing rapidly in this area, and the demands for whitewater releases from the Corps' reservoirs increase every year. A local group organized Whitewater Championships, Inc. (WCI) to promote whitewater sports. WCI hosted the 1989 World Whitewater Canoe/Kayak Championship on the Savage River.

The Savage River below the dam also is an outstanding trout stream. The project is regulated for the cold water fishery. As a result, about 5 miles of the river below the dam have become a trophy trout stream. Temperature control at Savage Dam is unique because the project has only a bottom release outlet works. The Baltimore District has successfully been able to managed the release temperature in an acceptable range for the fishery.

This paper discusses the mode of operation for downstream temperature control, the change of operation due to whitewater releases from 1987 to 1989, and its downstream temperature impacts.

## SAVAGE RIVER DAM

Savage River Dam is located in Garrett County, Maryland (Figure 1). The construction was completed by the Corps in 1952. The Upper Potomac River Commission operates and maintains the project, and the Corps regulates flow. The drainage area of the Savage River above the damsite is 104 square miles. The project provides 20,000 acre-feet of storage for pollution abatement, downstream hydropower, and water supply. There is no flood control storage. In 1988, whitewater sports was added as one of the project purposes (The Water Resources Development Act of 1988).

Savage Lake is regulated as a seasonal pool. It is filled to the spillway elevation (1468.5 NGVD) in spring, and gradually lowered in the summer and reaches elevation 1410-1420 NGVD in late fall due to augmentation flow for the North Branch Potomac River. Savage River Dam is regulated with Jennings Randolph Lake, which is a nearby Corps reservoir, in a system concept to improve water quality conditions on the North Branch Potomac River.

The lake is thermally stratified in summer (the maximum depth of the lake is 150 feet deep). Physical elements for controlling flows are a spillway and a bottom withdrawal outlet works. The bottom withdrawal outlet works consist of two service gates and a bypass gate. When stream flow drops in summer, the storage is used for supplying augmentation flow.

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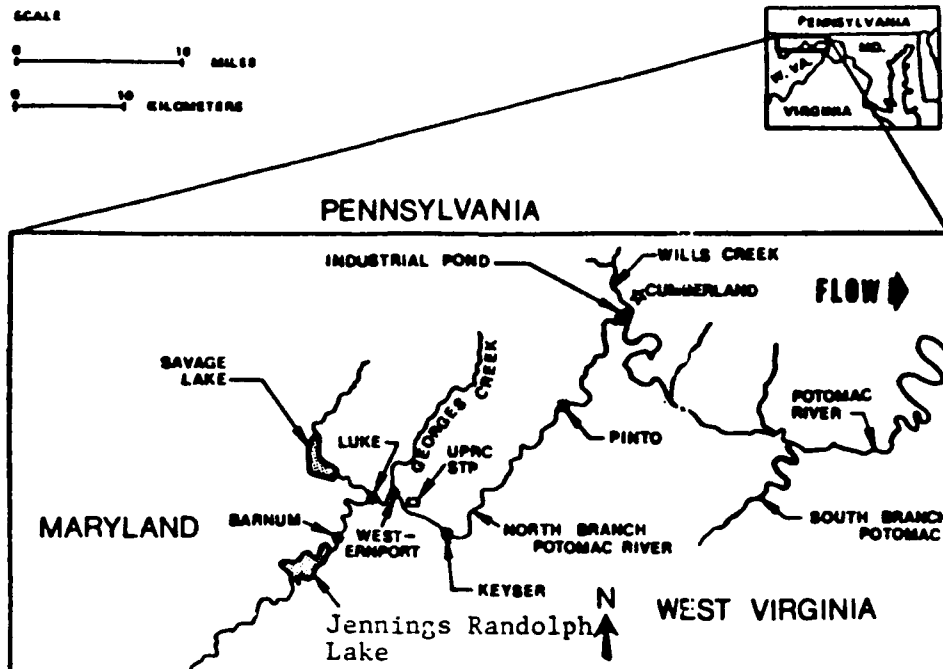
Figure 1. Area Map

## TEMPERATURE CONTROL AT SAVAGE RIVER DAM

The normal mode of operation at Savage Dam was to discharge through the service gates when the pool was below the spillway elevation. When the pool elevation was above spillway elevation, the service gates were fully closed, and all the discharges were made through the spillway. This mode of operation frequently created a thermal shock when the flow was switched from the bottom to the surface or from the surface to the bottom. The magnitude of the thermal shock was up to about 10-15 degrees C. The impact on the fishery habitats was disastrous. This operation had been applied until 1981, and at that time the fishery was very limited.

In 1982, the Corps initiated a new mode of operation to avoid spillway overflow. All the discharges were made through the service gates. This mode of operation evacuates most of the very cold waters on the bottom in the early spring and replaces them with warmer spring inflow. The pool is maintained about 2 feet below the spillway. This operation maintains a steady downstream temperature and prevents thermal shock.

Figure 2 exhibits the results of downstream temperature control in 1983. The outflow temperature gradually increased with time. The outflow temperature suddenly rose 4-5 degrees C on Day 140 because the all cold water was replaced with spring warm water due to high inflow. The outflow temperature was steady from Day 150 to Day 240 because inflow was low and steady. With the exhaustion of the cold water storages after Day 240, the outflow temperature gradually increased and reached a peak of 18 degree C on Day 290. This operation has succeeded in preventing thermal shock and provides adequate water temperatures for the trout fishery habitat. One disadvantage is that spring outflow temperature is colder than natural stream temperature.

Proper management of cold water storage is the key for success of downstream temperature ontrol. The Maryland Department of Natural Resources established a maximum stream temperature at the mouth to protect the trout.

CALENDAR DAYS 1983
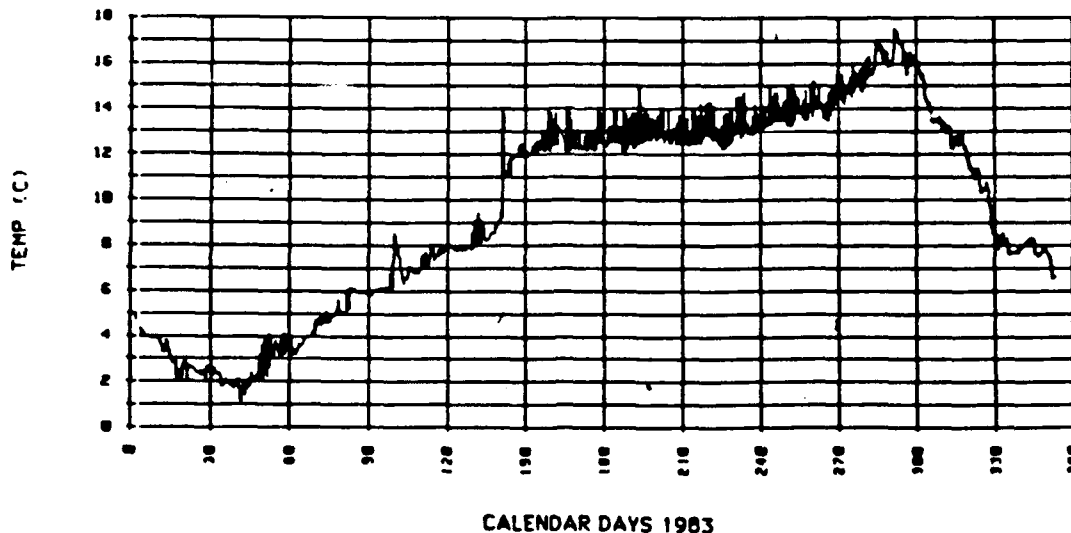
Figure 2. Water Temperature below Savage River Dam

The maximum temperature is 21 degreess C. The outflow volume and its temperature affect water temperature. Low outflow quickly warms up downstream temperature. Figure 3 exhibits the relationship with outflow volume and its temperature. For instance, when the outflow is 30 cfs and its release temperature is 18 degreess, the water temperature at the mouth reaches 21 degreess C. The release temperature often reaches 18 degrees C in September. When release temperature is above 18 degrees C, the outflow must be greater than 30 cfs. Generally, outflows are released in the range of 40-60 cfs during the low flow periods. Effective use of the cold water storage by optimizing outflow with time can achieve reasonable temperature control.

## TEMPERATURE CONTROL WITH WHITEWATER RELEASES

In 1984, WCI requested the Upper Potomac River Commission to release for about 15 days for the 1989 World Whitewater Canoe/Kayak Championships. The Upper Potomac River Commission, with the technical assistance from the Corps and Interstate Commission of Potomac River Basin, suggested that the 15 days of releases (estimated about 6,500 acre-feet of storage) under reasonable meteorological conditions was possible. The decision made in 1984 considered only water supply as no fishery had yet developed.

In 1987, WCI conducted canoe/kayak races as a part of training for the 1989 races. The releases were made twice, in June and in August. Each time about 2,200 acre-feet of storage were used. The releases were in the range of 850-1200 cfs. The flow went from 40-50 cfs to 850-1200 cfs within 2 hours, twice a day, during the races.

WCI requested about 4,500 acre-feet of storage to host the Maryland International Canoe/Kayak Classic on June 20-26, 1988. A study was conducted to determine whether the project could provide that storage without any adverse impact on trout. The problem was if the project could provide enough cold water after the releases to sustain the fishery for the rest of the summer. Savage Dam generally has 11,500 acre-feet of cold water storage in mid-June. Table 1 shows the availability of cold water storage with various outflows and different scenarios of whitewater releases. Based on these data, the maximum available storage for whitewater sports while releasing the minimum outflow of 40 cfs is 4,000 acre-feet.

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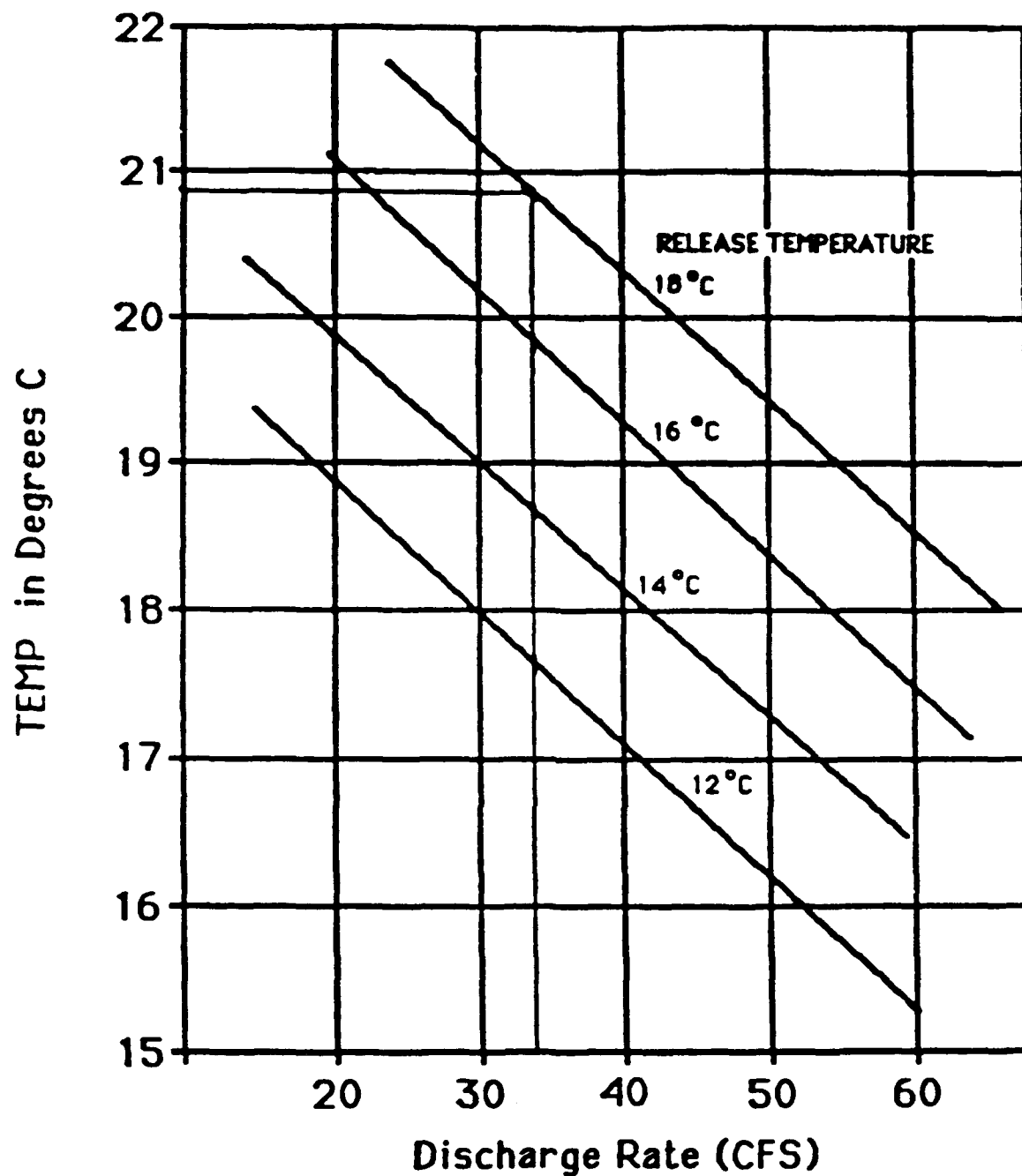
Figure 3. Expected Water Tempurature at the Second Bridge below the Dam

A total of 4,200 acre-feet of storage were used for the whitewater releases over seven days in 1988. As a result, the pool was lowered 15 feet (Figure 4). The cold water storage (below 18 degreess C) was 14,000 acre-feet before the whitewater releases and 8,500 acre-feet after the releases. About 800 acre-feet of cold water were warmed up during the event because of the large and unsteady rate of releases.

# TABLE 1

## Available Coldwater Storage With Whitewater Releases

| Whitewater Releases (ac-ft) | Outflow (cfs) | June (ac-ft) | July (ac-ft) | August (ac-ft) | September (ac-ft) |
|---|---|---|---|---|---|
| | 20 | 9,500 | 8,260 | 7,020 | 5,820 |
| | 30 | 9,500 | 7,640 | 5,780 | 3,980 |
| 2,000 | 40 | 9,500 | 7,020 | 4,540 | 2,140 |
| | 50 | 9,500 | 6,400 | 3,300 | 300 |
| | 60 | 9,500 | 5,780 | 2,060 | 0 |
| | 20 | 9,000 | 7,760 | 6,520 | 5,320 |
| | 30 | 9,000 | 7,140 | 5,280 | 3,480 |
| 2,500 | 40 | 9,000 | 6,520 | 4,040 | 1,640 |
| | 50 | 9,000 | 5,900 | 2,800 | 0 |
| | 60 | 9,000 | 5,280 | 1,560 | 0 |
| | 20 | 8,500 | 7,260 | 6,020 | 4,820 |
| | 30 | 8,500 | 6,640 | 4,780 | 2,980 |
| 3,000 | 40 | 8,500 | 6,020 | 3,540 | 1,140 |
| | 50 | 8,500 | 5,400 | 2,300 | 0 |
| | 60 | 8,500 | 4,780 | 1,060 | 0 |
| | 20 | 8,000 | 6,760 | 5,520 | 4,320 |
| | 30 | 8,000 | 6,140 | 4,280 | 2,480 |
| 3,500 | 40 | 8,000 | 5,520 | 3,040 | 640 |
| | 50 | 8,000 | 4,900 | 1,800 | 0 |
| | 60 | 8,000 | 4,280 | 5,600 | |
| | 20 | 7,500 | 6,260 | 5,020 | 3,820 |
| | 30 | 7,500 | 5,640 | 3,780 | 1,980 |
| 4,000 | 40 | 7,500 | 5,020 | 2,540 | 140 |
| | 50 | 7,500 | 4,400 | 1,300 | 0 |
| | 60 | 7,500 | 3,780 | 60 | 0 |
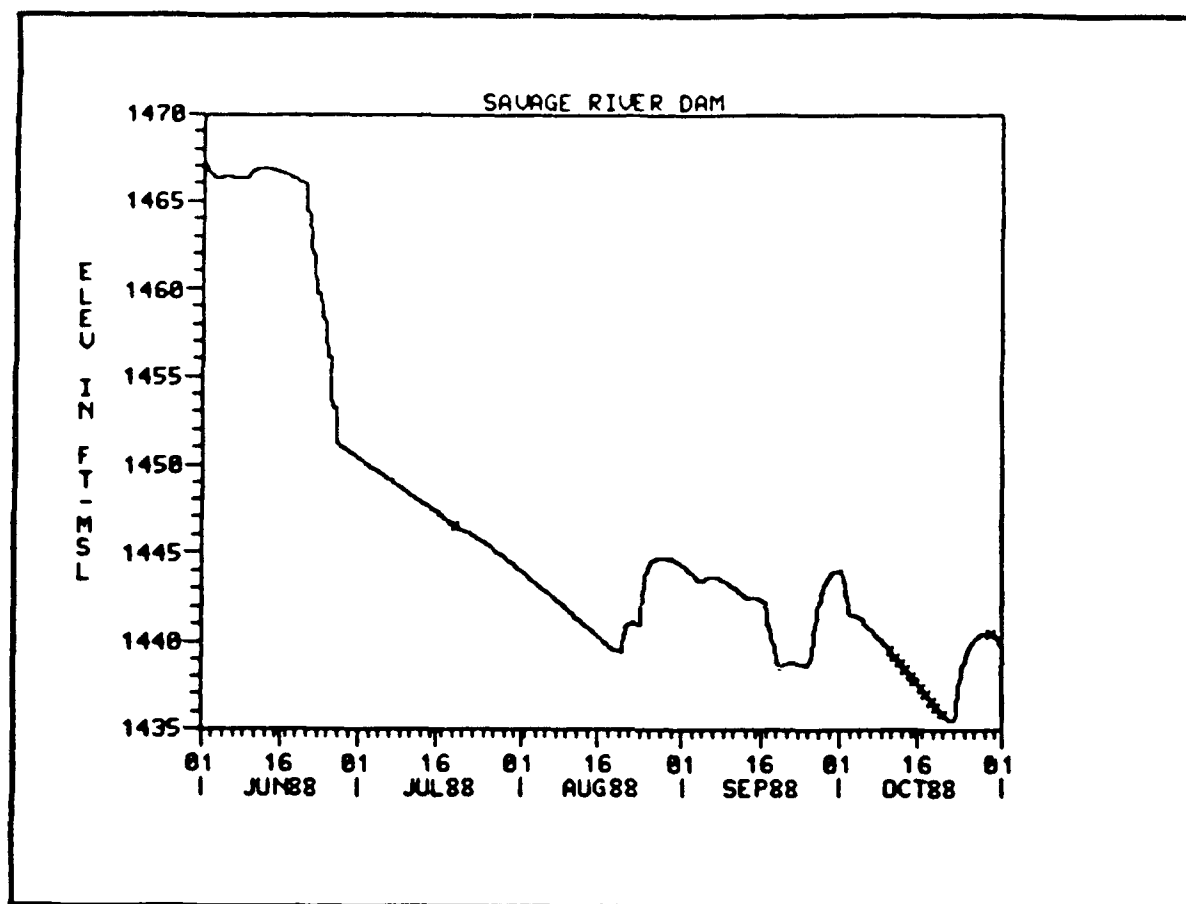
Figure 4. Savage Pool Elevation, 1988

Table 2 shows the cold water storage with time. It was a hot and dry summer until late August and cool in September. About 4,500 acre-feet of cold water were left on August 29 and 3,000 acre-feet on September 13. A minimum outflow of 34 cfs was maintained after the races until Day 240 (August 29). The outflow was 70 cfs after Day 240 because a period of cold inflow in late August recharged the cold water pool. About 1,000 acre-feet of the cold water storage were lost between June and September due to lake heating.

Figure 5 exhibits pool elevations without whitewater releases in 1985, and Figure 4 exhibits the pool elevations with whitewater releases in 1988. The whitewater releases caused the severe storage loss at the beginning of the drought. Therefore, the rest of storage is used only for maintaining the minimum outflow. Without whitewater releases, the storage might be used for maintaining augmentation flow during the drought periods. Figures 6 and 7 exhibit inflow and downstream temperatures. Inflow temperature was 21 to 23 in the summer and below 18 degreess C after Day 240. The downstream temperature was 11 degreess C at the beginning of the races and reached the peak (18 degreess C) around Day 260. If September's weather was hot and dry, which is not unusual, the cold water would have been exhausted in September (Day 260) and the project would have encountered downstream temperature control problems in late September and October.

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## TABLE 2

### Cold Water Storage With Time

| DATA | POOL ELEVATION | THERMOCLINE (ft) (ABOVE 18° C) | STORAGE (ac-ft) | | | | |
|------|----------------|--------------------------------|-------|---------------|---------------|----------------------|-------------------|
| | | | TOTAL | WARM WATER | COLD WATER | DISCHARGE (STORAGE) | INFLOW (STORAGE) |
| 6/20 | 1466.3 | 15 | 19,300 | 5,100 | 14,200 | | |
| | | | | | | 4,600 | 140 |
| 6/27 | 1451.1 | 22 | 14,200 | 5,300 | 8,900 | | |
| | | | | | | 970 | 204 |
| 7/11 | 1448.2 | 24 | 13,300 | 5,600 | 7,700 | | |
| | | | | | | 960 | 220 |
| 7/25 | 1445.3 | 27 | 12,300 | 5,600 | 6,700 | | |
| | | | | | | 910 | 140 |
| 8/8 | 1442.2 | 30 | 11,800 | 6,100 | 5,700 | | |
| | | | | | | 1,370 | 2,230 |
| 8/29 | 1444.67 | 40 | 12,900 | 8,400 | 4,500 | | |
| | | | | | | 1,250 | 811 |
| 9/13 | 1442.5 | 50 | 11,800 | 8,800 | 3,000 | | |
| Total | | | | | | 10,060 | 3,740 |

Use of cold water storage from 20 June - 13 September

        Balance in the lake:  14,200 - 3,000 = 11,200 ac-ft
        Total discharge:        10,060 ac-ft

Accumulation warm water from 20 June - 13 September

        Balance in the lake:  8,800 - 5,100 = 3,700 ac-ft
        Total inflow:           3,740 ac-ft
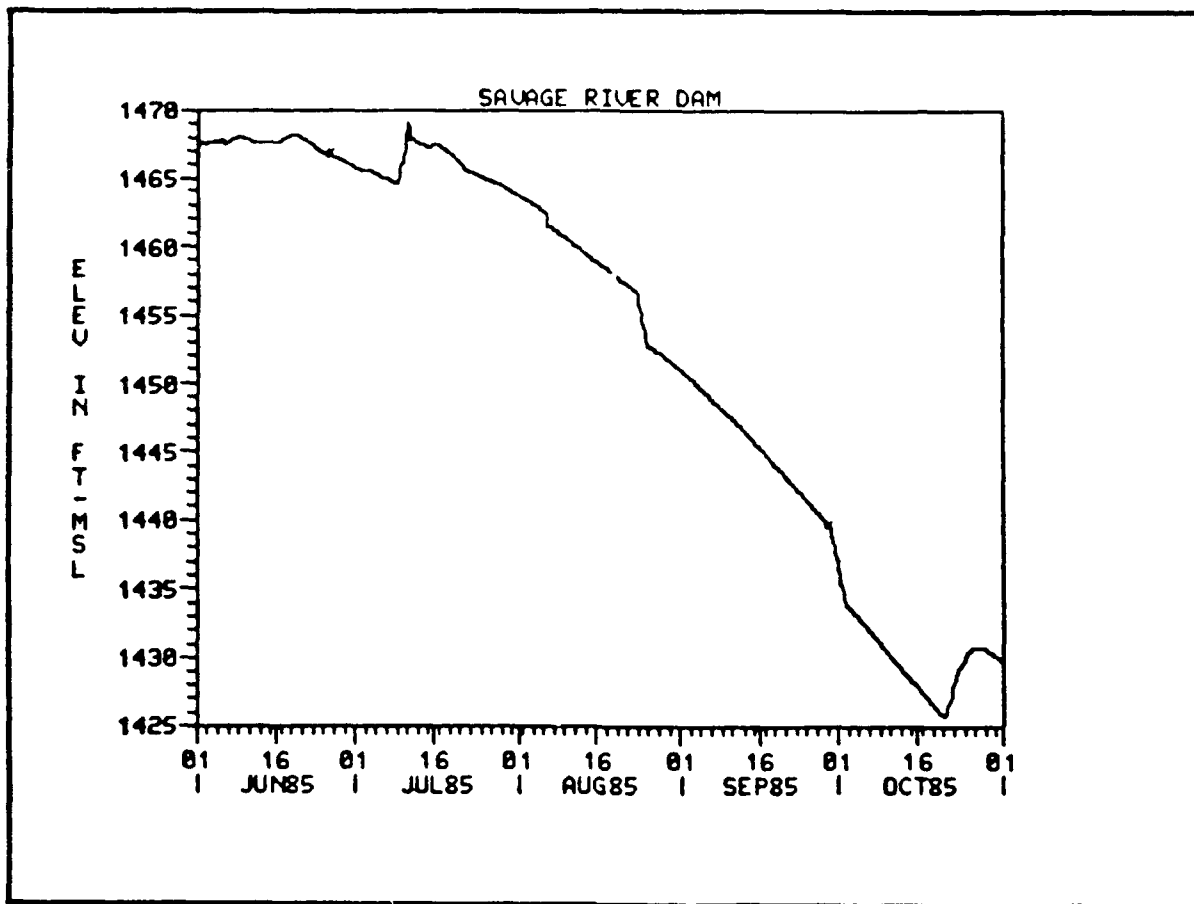
Figure 5. Savage Pool Elevation, 1985

About 8,200 acre-feet of storage, equivalent to 40% to the total storage, was requested for the 1989 event. Releasing 8,200 acre-feet of storage in June causes significant adverse impacts on the trout and the authorized purposes of the project. The Corps offered a maximum of 6,500 acre-feet for the races. The use of this storage in June would still result in unfavorable water temperature (above 21 degrees C) at the mouth in September and October if the present mode of operation was applied. Several alternative modes of operation were discussed with the Maryland Department of Natural Resources to resolve the potential for fish kills due to high water temperature. The selected mode of operation was to allow spillway overflow (old operation plan) to provide maximum cold water storage before the races.

In 1989, the pool reached the spillway in early May. All discharges were made over the spillway until race day. About 15,000 acre-feet of cold water storage were available before the race. A total of 7,000 acre-feet of storage were released during the fourteen days of the races.

Figures 8 and 9 exhibit the 1989 inflow and downstream temperatures. Outflow temperature had rapidly warmed up once the outflows were released only over the spillway after Day 135. The outflow temperature reached 21 degreess C before the races. To alleviate thermal shock on the first day of the races, the service gate was partially opened for the five days to blend top and bottom releases to radually lower the release temperature as spillway flow subsided. The outflow temperature before the races gradually dropped for the five days, and reached 6 degreess on the first day of the races when the outflow was discharged only through the service gate. The outflow temperature was 8 degrees C at the end of the races. After the races, Day 177, the outflow temperature was almost constant 8-10 degreess C until Day 195. This temperature was 3-4 degreess cooler than that with the normal operation because the modified spring time operation had maintained cold water storage before the races.

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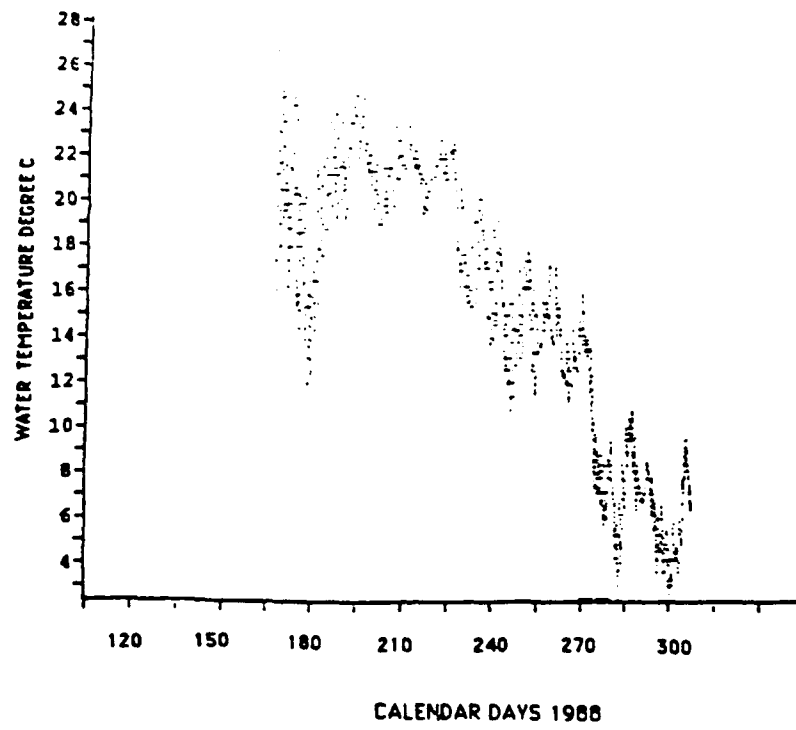
Figure 6. Inflow Temperature, 1988
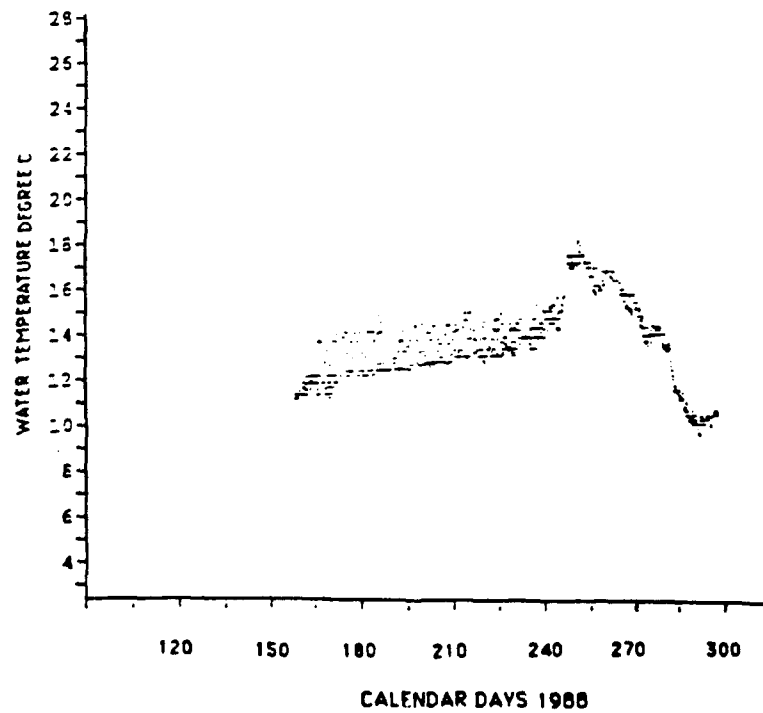


Figure 7. Water Temperature at the Second Bridge below the Dam

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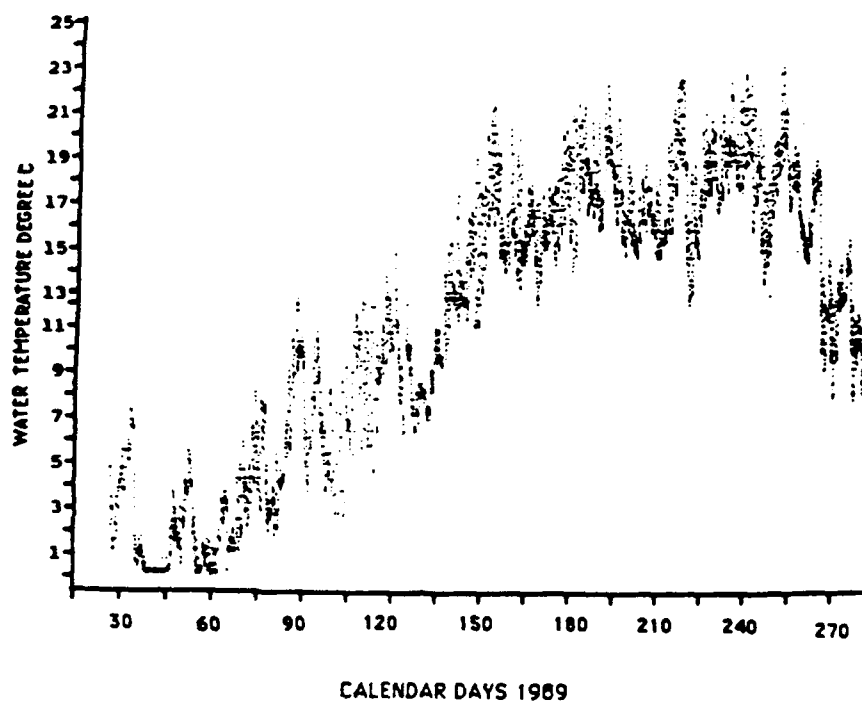
Figure 8.  Inflow Temperature, 1989
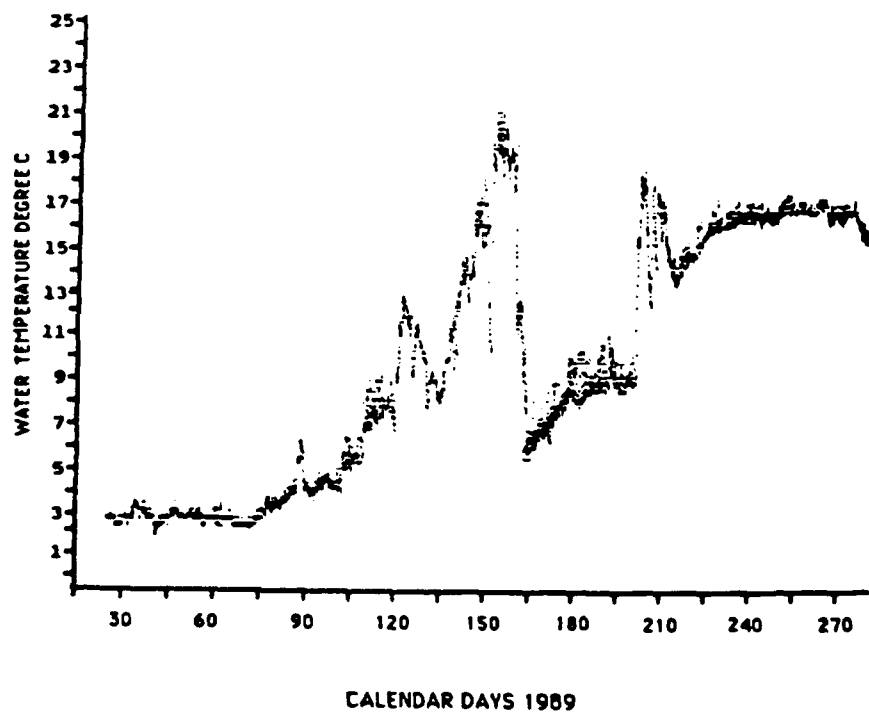


*Figure 9.  Water Temperature at the Second Bridge below the Dam*

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The summer of 1989 was wet and cool. Inflow temperature in the summer ranged from 17 to 21 degreess C, which is well below the average inflow temperature. The pool refilled with cool inflow due to high runoff in July and began spilling on Day 195. The releases were made by both the spillway and service gate for 22 days (Day 195 to Day 217). The reason both the service gate and the spillway were used, was to evacuate the now excess of very cold ater and to replace it with the cool inflow. The spillway flow was maintained to conserve the hypolimnetic water of 15-17 degreess C to substain the fishery for the remainder of the summer. After Day 218 all the discharges were made through the service gates with the outflow temperature at 15-17 degreess C.

The outflow temperature in 1989 showed two thermal shocks (Day 155 and Day 195). Figure 10 exhibits the 1989 storage availability of cold water. The lake had an adequate cold water storage because it was the wet and cool summer of 1989. The downstream temperature was maintained below 21 degreess C at the mouth throughout the year.
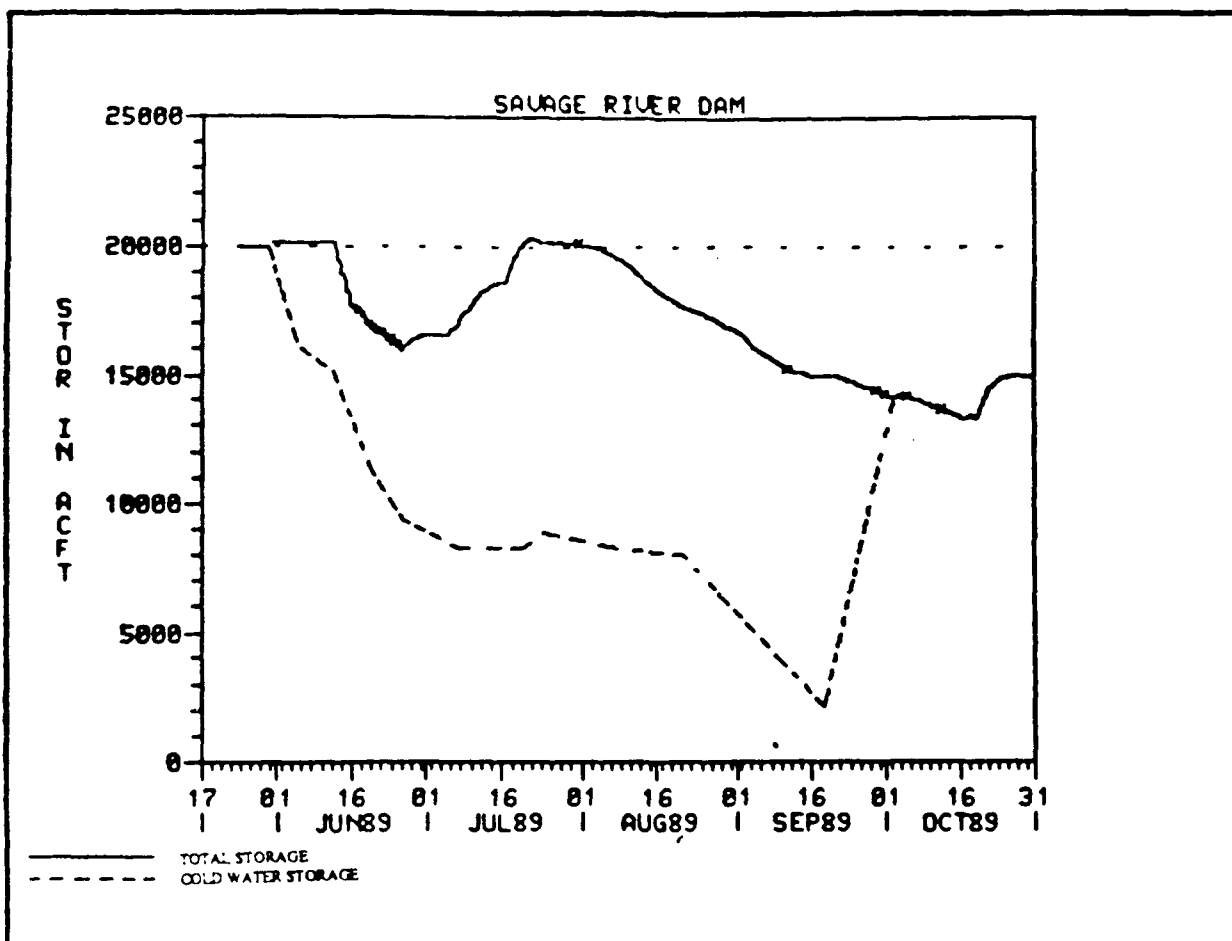


Figure 10. Total Storage vs. Cold Water Storage With Time

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## SUMMARY

A unique new method for controlling downstream temperature without a selective withdrawal system is employed at Savage River dam. The present mode of operation has regulated downstream temperature to provide adequately controlled downstream temperature for the cold water fishery. The Savage River below the Dam is considered the best trout fishery in Maryland, and one of the best on the East Coast. Also, the Savage River is one of the best places for whitewater sports.

The temperature control encountered difficulty when large volume whitewater releases were made in June or July. If the large volume whitewater releases were made, the mode of operation was changed Uto manage cold water. This operation resulted in downstream temperatures which were too cold in July and created thermal shocks. The best time for whitewater releases from the Savage River Dam is in late August or September when temperature control is no longer critical.

If the use of the storage for whitewater sports is limited, the Savage River Dam project can successfully support both the trout and whitewater sports.

## REFERENCE

U.S. Army Corps of Engineers. 1985. "Master Manual for Reservoir Regulation North Branch Potomac River Basin," U.S. Army Engineer District, Baltimore, Maryland.

# IMPROVING DISSOLVED OXYGEN IN THE RELEASES
## FROM TABLE ROCK DAM, MISSOURI

by

Stacy E. Howington[1]

Table Rock Dam is located in southwestern Missouri at mile 528.8 on the White River. The average annual flow at the dam site for a 28-year record is 4,400 cfs. The dam contains 4 main hydropower units whose cumulative capacity is about 200 MW (U.S. Army Engineer District (USAED), Little Rock 1985). The receiving water for Table Rock Dam releases is Lake Taneycomo, which provides a cold-water habitat for trout. Table Rock release dissolved oxygen (DO) concentrations have been observed during the summer and fall to be less than the Missouri state standard of 6.0 mg/l. Low release DO is also a problem during non-generation due to gate leakage estimated to be as much as 95 cfs (USAED, Little Rock 1985).

The Little Rock District published a summary report of the DO studies in Table Rock Lake and Lake Taneycomo (USAED, Little Rock 1985) in which the most attractive alternative of those examined was the construction of selective withdrawal towers to front two of the four existing hydropower units. These modified units were to withdraw epilimnetic water to mix with the hypolimnetic water from the other two units to achieve a higher composite release DO. A study conducted by Wilhelms and Hamlin (1988) indicated that, although the proposed selective withdrawal structures would produce withdrawal from the epilimnion only, the desired 6.0 mg/l composite release DO could not be attained every day of the year while maintaining release temperatures below a maximum criterion of 17° C using historical operations. Since selective withdrawal proved to be an inadequate solution to the DO problem at Table Rock Dam, a subsequent study was undertaken to reexamine other release DO improvement techniques.

## POTENTIAL IMPROVEMENT TECHNIQUES

Many alternatives for increasing dissolved oxygen in dam releases are available and the Little Rock District (USAED, Little Rock 1985) has evaluated several. The following is a list of commonly considered alternatives that are described in more detail by Wilhelms et al. (1987):

a. lake destratification,
b. hypolimnetic aeration,
c. hypolimnetic oxygenation,
d. localized mixing,
e. turbine venting,
f. self-aerating turbines,
g. supplemental spillway releases,
h. penstock oxygenation, and
i. tailwater systems.

The large hydropower flows and desire for cool releases permit the dismissal of several alternatives at Table Rock. Lake destratification would unacceptably warm the releases. Localized mixing would warm the releases somewhat and, with the large flow rates through the structure, might result in accidental lake destratification. Although a properly working hypolimnetic aeration system would not warm the releases, the large flows would require substantial amounts of air and could cause lake destratification. Nitrogen supersaturation of the releases when injecting air at these depths would

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[1]Research Hydraulic Engineer, U.S. Army Engineer Waterways Experiment Station.

also be a concern. The large flows would also limit the effectiveness of most tailwater systems. Additionally with these systems, any unsatisfied oxygen demand in the release waters would produce an unacceptable DO sag within Lake Taneycomo.

Other alternatives can be excluded by the low DO gate leakage. Turbine venting would not help with this problem. Nor could turbine venting alone achieve the occasionally large DO uptake required for Table Rock (Wilhelms et al 1987). However, this alternative may be considered as a supplemental measure to another alternative. Self-aerating turbines are at the early stages of research and are not yet available. Even after they become available, they may encounter a similar DO uptake limitation as today's turbines.

Supplemental spillway releases are presently being used at Table Rock Dam. These flows bypass the hydropower units, warm the releases, and are insufficient to achieve the desired DO for all conditions.

Hypolimnetic oxygenation could provide the desired DO in the releases for both operation and gate leakage without warming the release water. Penstock oxygenation might provide the required DO as well. However, this system's contact time, and therefore its gas transfer efficiency, is less than for hypolimnetic oxygenation. An undesirable downstream DO sag is also possible with penstock injection. Still, penstock injection may be effective in conjunction with another method by acting as an emergency or supplemental technique.

HYPOLIMNETIC OXYGENATION

Hypolimnetic oxygenation has been used successfully at Richard B. Russell (RBR) Dam on the Savannah River, a project similar in several means to Table Rock. The depths are comparable, the flows are large, the state standards for DO are the same, and the release DO concentrations, without remedial measures, are significantly deficient.

One item needed to properly size a hypolimnetic oxygenation system is the maximum hypolimnetic oxygen demand. The demand can be simply estimated by determining the total hypolimnetic oxygen content for two times and dividing by the elapsed time to produce the average DO decay rate over that period. The hypolimnetic DO concentrations from observations in a normal year, a dry year, and a wet year, as well as observations and predictions from the work of Wilhelms and Hamlin (1988), were examined. From these data, the steepest decline in DO occurred as the hypolimnetic oxygen content decreased from 8.9 million kg to 3.5 million kg in 26 days. The resulting decay rate, 0.111 mg/l/day, may be an underestimation of the demand during aeration since demand has been observed to increase during aeration (Holland and Tate 1984). The demand computed for Table Rock corresponds well with the 0.1039 mg/l/day demand computed independently and with another technique for Beaver Lake[2] immediately upstream of Table Rock Lake.

HYPOLIMNETIC OXYGENATION SYSTEM LOCATION

As was done at RBR, the proposed oxygenation system at Table Rock would be divided into two pieces. One would be designed to provide oxygen in the far-field and the other, the near-field. Operation of both parts at RBR was found to provide the best efficiency. The near-field structure at RBR is located immediately in front of the hydropower intakes. A similar location would be tentatively adopted at Table Rock. The far-field system at RBR is about 1 mile upstream of the dam. Two potential sites were identified for the location of the far-field system at Table Rock. The far-field system should be located adequately far upstream such that some storage of oxygen in the reservoir is affected. This

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[2]Unpublished data, CPT Ed Meyer, U.S. Army Engineer, Waterways Experiment Station.

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provides a time lag between a system failure and the release of poor DO water which might be advantageous in the repair of malfunctioning equipment. It also provides ample contact time for maximum oxygen uptake and the exhaustion of latent biological oxygen demand prior to release so that a downstream DO sag can be minimized. This method also allays concerns about low DO gate leakage since DO in the hypolimnion will be continuously acceptable.

The first far-field system location identified for Table Rock (site A) was approximately one mile upstream of the project. This is near a peninsula which might provide access. The Little Rock District proposed another site (site B) for the far-field system that was about 3.1 miles upstream of the dam. This site would be advantageous since the Corps of Engineers already holds access to the reservoir at this point (Figure 1), and since it coincides with a natural constriction in the reservoir.
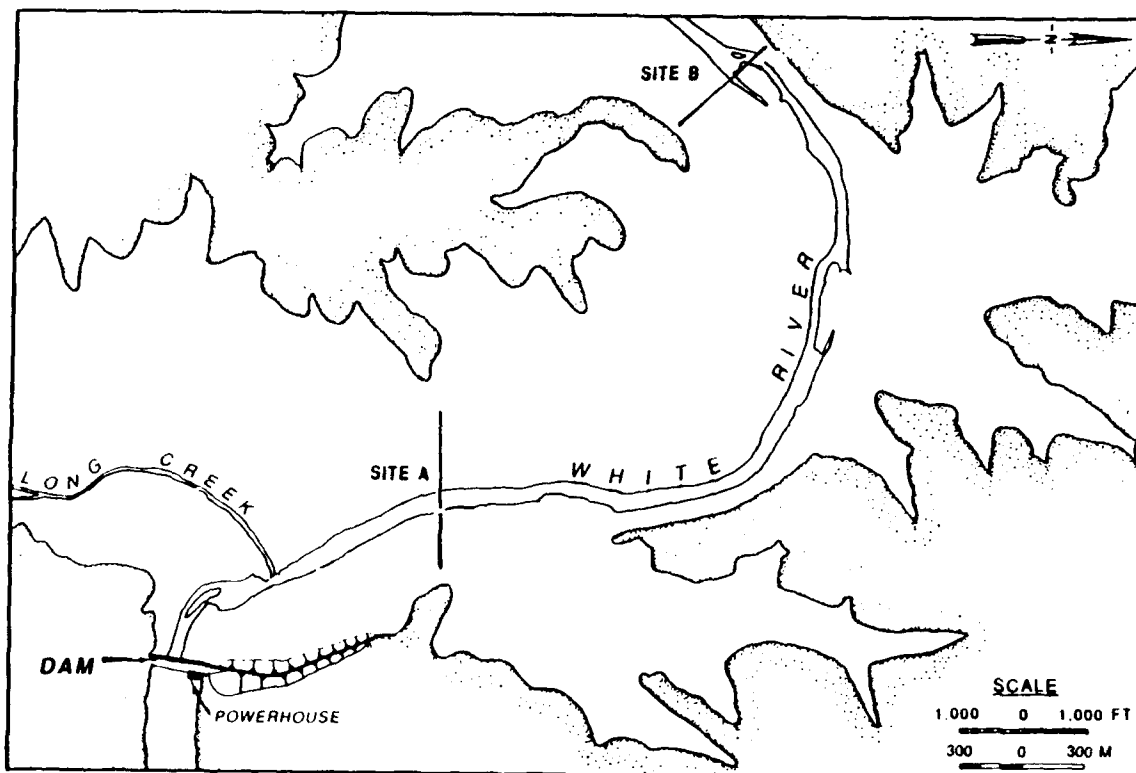


Figure 1. Potential locations of far-field diffuser systems.

In determining the travel times associated with these two locations, reservoir volumes between each of the proposed sites and the dam were calculated from contour maps. According to the Little Rock District, Long Creek, which enters Table Rock Lake near the dam (downstream of both sites A and B) provides virtually no flow compared to the White River[3]. Moreover, only a small amount of oxygenated water is expected to migrate into the Long Creek embayment. Therefore, the hypolimnetic volumes

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[3]Personal communication on 5 January 1989 with Mr. Gordon Bartelt, U.S. Army Engr., Dist., Little Rock, Little Rock, AR.

determined in computing travel times did not include the full extent of Long Creek, but were limited to the White River only.

## DISSOLVED OXYGEN DEFICIENCY CURVE

Since the release DO from the dam will vary from year to year, a general description of the dissolved oxygen deficiency for a worst-case DO year was needed to approximate maximum yearly oxygen requirements. The Little Rock District provided a DO deficiency design curve based on analysis of observed DO profiles (USAED, Little Rock 1985). This worst-case deficiency curve was adopted with minor adjustments to include additional data. The curve lies beneath the state standard from day 152 (June 1) to about day 349 (December 15) (Figure 2).
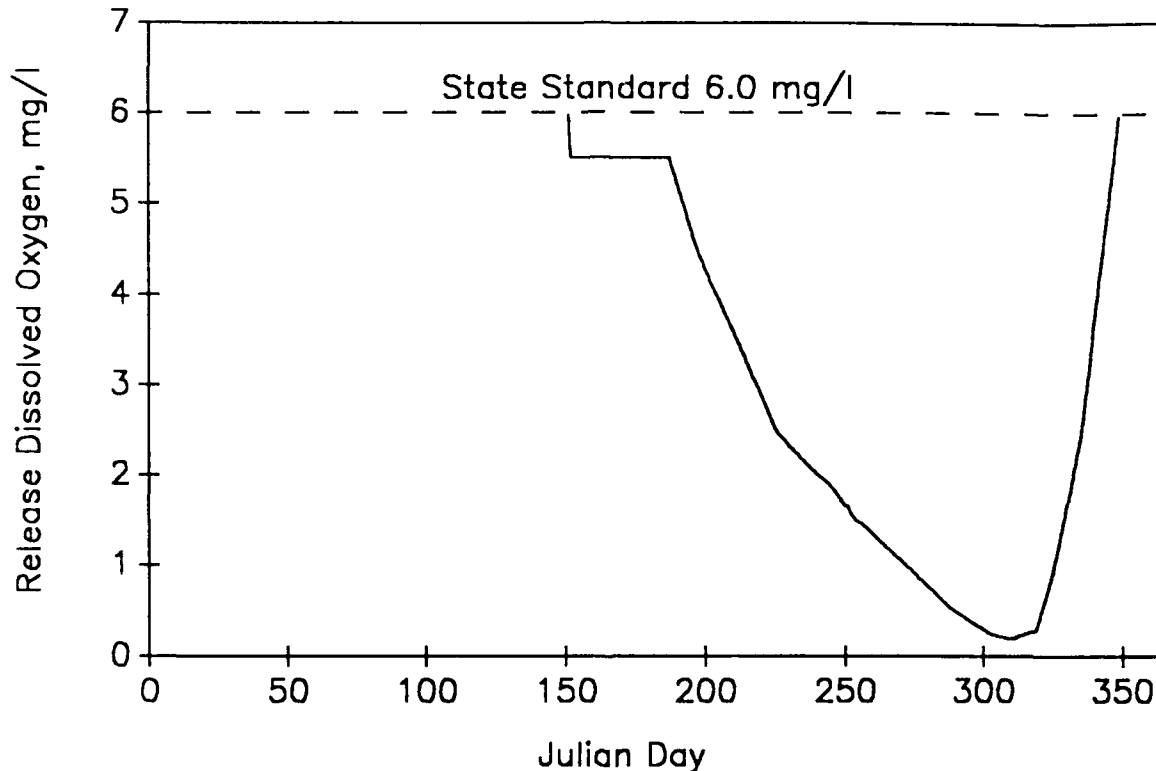
Figure 2. Adopted DO deficiency design curve.

## HYPOLIMNETIC OXYGENATION SYSTEM SIZING

Sizing of a hypolimnetic oxygen injection system is dependent on a maximum single-day oxygen requirement. Using the maximum DO deficiency and the maximum flowrate to determine the maximum delivery rate would result in a drastically over-designed system. Therefore, as was done for RBR, the design of the far-field system was based on a 10 percent exceedance of the daily average hydropower releases. Based on the seven years of average daily hydropower flows provided by the Little Rock District, the 10 percent exceedance is about 7,000 cfs. These flows, however, reflect the use of a restricted generation schedule.

The lowest prolonged penstock DO concentration from Figure 2 was about 0.2 mg/l during late November. Therefore, the largest oxygen deficit in the release water was 5.8 mg/l. Assuming initially that all the oxygen is added at the far-field system, the DO decay incurred as the water travels from the upstream diffusers to the dam must be added as a deficit. The decay-induced deficit was computed daily based on travel times computed from a 7-year (1982-1987) average flow for each day and the hypolimnetic volumes from sites A and B. The 0.111 mg/l/day decay was applied to these times to produce the additional deficit. The design deficits for sites A and B, which combine the maximum oxygen deficit and the travel-time-related decay deficit, were 6.24 mg/l and 6.88 mg/l, respectively. The system sizing, expressed as a maximum single-day delivery rate, is computed as (USAED, Savannah 1981):

$$O_2 \text{ (design)} = \frac{Q * \gamma_{O2} * DD * 86{,}400 \text{ sec/day}}{E * 2000 \text{ lbs/ton} * 10^6}$$

where

$O_2$ = maximum single day delivery rate, tons/day,
$Q$ = design discharge (7,000), cfs,
$\gamma_{O2}$ = specific weight of water (62.38), $lb/ft^3$
$DD$ = design DO deficiency, ppm
$E$ = design gas transfer efficiency (0.75).

The 75 percent transfer efficiency also came from the RBR system design. The RBR system has achieved efficiencies both higher and lower than 75 percent, but this was established as a reasonable number for design[4]. The efficiency is highly dependent on the cleanness of the diffuser heads.

Using the computation procedure in the previous paragraph, the maximum single-day oxygen requirement is approximately 157 tons for site A and 173 tons for site B. The RBR far-field system was designed for 150 tons/day maximum. If the Table Rock system were, indeed, this similar in size to the RBR system, and the same diffuser types were employed, the distribution system at Table Rock would require very nearly the same number of diffusers as at RBR.

Sizing of the near-field system would be done very similarly. The maximum daily delivery rate would be the same or a little less than for site A (the hypolimnetic oxygen demand is no longer considered). However, it would need to deliver, in event of the far-field system failure, the worst case DO deficit (5.8 mg/l) at 18,000 cfs over a short generation period. This calculation results in a system that can deliver 15.6 tons/hour.

## CRYOGENIC PLANT CONSTRUCTION VERSUS OXYGEN PURCHASE

Oxygen can either be bought and delivered to the site or it can be produced on-site with a cryogenic facility. Since RBR is a new reservoir, the oxygen demand is expected to change as the lake evolves and the decision regarding the cryogenic facility has not yet been made. The cost of purchasing and trucking-in liquid oxygen has been far less than was anticipated. Estimates during design of the facility had put the cost at $75-$100 per ton. However, costs since installation have been nearer $45-$55 per ton.

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[4]Personal communication on 23 January 1989 with Mr. Gary Mauldin, U.S. Army Engineer, Division, South Atlantic, Atlanta, GA.

The break-even point for constructing a cryogenic facility versus trucking-in the oxygen was approximated for RBR, based on 1981 dollars, to be about 5,300 tons/year (USAED, Savannah 1982b). This analysis pitted the annualized capital cost differential between the two alternatives against the yearly cost of oxygen at an estimated price of $100 per ton. Assuming the same capital costs and interest rates used in the RBR evaluation, and a more realistic cost of oxygen ($55/ton), the break-even point for plant construction would significantly increase to 9,600 tons per year. The choice of alternative for Table Rock will depend largely on the anticipated cost of oxygen in the area and the site location of the far-field system.

## ANNUAL OXYGEN REQUIREMENTS

The amount of oxygen on a yearly basis can be estimated by multiplying the daily averaged hydropower flows by the oxygen deficiency curve from Figure 2 and adding the travel-time-related decay. The oxygen requirements estimated by this procedure would be an overestimation of the average year's requirements. Although the hydropower flows represent seven-year averages for each day, the oxygen deficiency curve is virtually a worst-case scenario, and the travel-time-decay applies the largest observed DO demand at all times. Using these procedures, the approximate annual oxygen requirement would be 7,983 tons for site A and 11,182 tons for site B. These translate into an average of 40.5 tons per day for site A and 56.7 tons per day for site B during the 197 days of operation each year from June 1 to December 15. This represents a 40 percent larger annual oxygen requirement with the use of site B as compared to site A. These calculations assume that only the far-field system is operating. Experience with the RBR system has shown the most efficient mode of operation to be a combination of both systems simultaneously.

## TURBINE VENTING AS A SUPPLEMENT

For this study, turbine venting was only to be considered without restrictions on hydropower operation. Since 1971, the turbines' air vents have been blocked open and hydropower operation restrictions have been in place at Table Rock Dam to enhance the turbine's venting abilities. From field studies (USAED, Little Rock 1972), a small percentage of the incoming DO deficit will be satisfied with the turbine vents blocked open. Although turbine venting alone will likely not produce the desired 6.0 mg/l release DO, if used as a supplement to another method, some oxygen uptake and an associated cost savings could be had. A major concern, however, with turbine venting is a slight decrease in turbine efficiency. For the 1971 venting tests at Table Rock, this loss was determined to be one percent by one measurement technique and revised to a negligible amount by another technique (USAED, Little Rock 1972).

If a 10 percent DO deficit satisfaction might be gained through venting, the system sizing requirement for sites A and B could be reduced to 141 tons/day and 158 tons/day, compared to 157 tons/day and 173 tons/day, respectively, from the original system requirements. Annual oxygen consumption estimates would correspondingly decreased to 6,357 tons from 7,983 tons for site A and to 8,910 tons from 11,182 tons for site B. These types of improvements must be weighed against any decrease in power generation due to efficiency loss.

## RECENT FIELD STUDY

Recently the Environmental Laboratory of the USAE Waterways Experiment Station conducted a field exercise to more thoroughly examine the hypolimnetic oxygen demand, the travel times from sites A and B, and the exchange with Long Creek. The results of this study and a non-restricted hydropower schedule are being used to re-examine the sizing and delivery rate for the hypolimnetic oxygenation system.

## SUMMARY AND CONCLUSIONS

Of the widely know alternatives for improving release DO from reservoirs, hypolimnetic oxygenation has demonstrated the highest potential for meeting the Missouri state standard for DO (6.0 mg/l) while not violating the temperature criterion for the cold-water fishery below Table Rock Dam. Variations of this alternative include:

    a. in-lake near-field and far-field diffuser systems,
    b. in-lake far-field diffusers and a penstock diffuser,
    c. either a. or b. with turbine venting as a supplement.

A stand-alone hypolimnetic oxygenation system similar to the one operating at Richard B. Russell Dam in the Savannah District was examined. The system would have both a far-field and a near-field component. Based on the DO deficiency and flow information available, a hypolimnetic oxygen system, operating alone, would need to be similar in size to the one at RBR to meet the required release DO. The design delivery rate should be about 150 to 175 tons of oxygen per day, depending on the site selection for the far-field system.

Two potential sites were identified for location of the far-field portion of the hypolimnetic oxygenation system: Site A which is 1 mile upstream of the dam, and Site B which is 3.1 miles upstream of the dam and at a CE-owned access point. Using the conservative assumption that the maximum observed DO demand in the hypolimnion will always exist, the added costs of using site B will be substantial. From the hypolimnetic-oxygenation only alternative in Table 1, the additional oxygen requirement associated with selecting site B over site A is about 3,200 tons per year. At $55 ton, that is an additional $176,000 annually for the worst-case comparison.

Using a penstock oxygenation system in place of the near-field system like that at RBR may be a viable alternative. Insufficient data exist, however, for the thorough evaluation of a penstock oxygenation system. It can be rationalized that the gas transfer efficiency of a penstock oxygenation system will be less than an in-lake oxygenation system, but so will the capital costs. A better estimation is required of the oxygen transfer efficiencies, potential turbine efficiency losses, and possible modifications required to install an effective penstock injection system.

Although turbine venting alone will not meet the release DO requirements, it may provide a very attractive means of satisfying a small part of the deficit, regardless of which other alternative is selected.

Several conclusions and recommendations arise from this study:

    a. The potential cost savings associated with the use of site B compared to site A for a far-field hypolimnetic oxygenation system should be thoroughly examined since the system location appears to have significant monetary implications.

    b. Turbine venting may be an attractive means of "capping" the oxygen level of the hydropower releases.

    c. A field study should be conducted to accurately assess the gas transfer efficiency of penstock oxygen injection at Table Rock. This would provide the needed information to separate the remaining alternatives.

    d. A pilot study of the entire system should be conducted prior to finalization of the design. The costs of such a study would be warranted by the potential savings in system sizing and site selection information.

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# PHYSICAL PROCESSES AFFECTING REAERATION AT LOW-HEAD WEIRS

by

Steve Wilhelms[1] and John Gulliver[2]

## BACKGROUND

Low-head hydraulic structures are generally associated with navigation or water supply projects. These structures are usually "run-of-the-river" and have the objective of maintaining a constant upstream pool elevation. As a consequence, the hydraulic structures at these projects many times consist of a gated sill or low-head spillway and a fixed- or adjustable-crest weir. As water passes over or through any hydraulic structure, the opportunity for reaeration (or more generally, gas transfer) exists. In fact, the gas transfer that would normally occur in several miles of river flow can occur in the short residence time of the turbulent flow conditions of a hydraulic structure.

In the past, the focus of interest in this gas transfer has been reaeration, the absorption of oxygen into the water. More recently, however, the desorption of volatile organics or toxics that may be dissolved in the water has become important. Thus, a thorough understanding of the physical processes affecting gas transfer becomes even more significant because the physical processes that enhance oxygen absorption usually increase the transfer of any dissolved volatile.

The effects of the important processes must be quantified into the mathematical description of gas transfer, which is presented below. The rate of change of a volatile compound's concentration due to air-water transfer can be given as

$$\frac{dC}{dt} = \frac{k_L A}{V} (C_s - C) \tag{1}$$

where $C$ is concentration, $C_s$ is *saturation concentration*, $k_L$ is the liquid film coefficient, $A$ is air-water surface area, and $V$ is the control volume corresponding to the surface area.

If it is assumed that $k_L A/V$ remains constant over a given hydraulic structure, Equation 1 may be integrated and rearranged to give a transfer efficiency $E$, originally proposed by Gameson (1957):

$$E = \frac{C_d - C_u}{C_s - C_u} = 1 - \exp\left[-\frac{k_L A}{V} t\right] \tag{2}$$

where $C_u$ is concentration upstream of the structure, $C_d$ is concentration downstream of the structure, and $t$ is the residence time between the upstream and downstream locations.

A transfer efficiency of 1.0 means that full transfer up to the saturation value has occurred at the structure. No transfer would correspond to $E = 0$. The transfer efficiency is simple to compute from

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field measurements and can also be transferred between compounds and adjusted for temperature (Gulliver, et al. 1990). It is difficult to predict $\Xi$, however, because the values of $k_L$ and $A$ are not known for hydraulic structures.

## PHYSICAL PROCESSES

The physical processes that affect gas transfer at a weir should be governed by the flow conditions or hydraulic conditions at the structure. Consider flow over a fixed-crest weir shown in Figure 1. Intuitively, it seems logical that the following physical processes would affect gas transfer:

a. turbulent mass transfer at the water surface and within the body of the flowing water, similar to any other flow situation, such as a river or stream,

b. mass transfer across the surface of air bubbles entrained along the free surface of the nappe and at the plunge point of the nappe,

c. mass transfer enhancement by pressure (bubbles under hydrostatic pressure of tailwater)
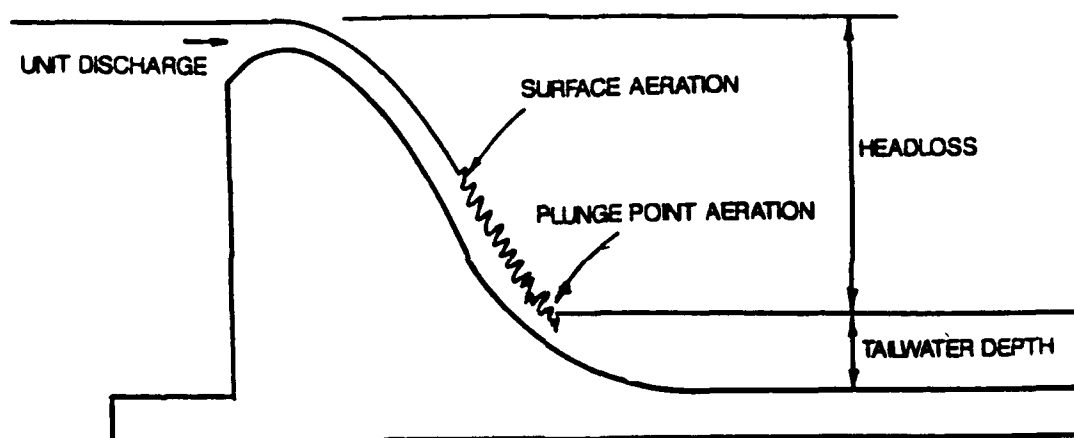


Figure 1. Typical Overflow Spillway

The problem becomes one of characterizing the effects of these processes with parameters that are measurable and directly relate gas transfer to the physical process.

Consider first the turbulent mixing process. It seems logical the rate of turbulent mixing would significantly affect gas transfer because of the concept of water-surface renewal (water surface that is swept away from the surface and "renewed" with water from below) causing increased gas transfer, and because of turbulent mixing within the water body. In fact, Danckwerts (1951) surface renewal model is generally believed to be the most applicable to a parametric description of the liquid film coefficient.

$$k_L = \sqrt{Dr}$$

where $D$ is molecular diffusivity and $r$ is the average surface renewal rate.

Difficulty arises in applying this model because an accurate parametric description of r has not been achieved for open channels (Wilson and MacLeod, 1974; Gulliver and Halverson, 1989), especially those with a velocity and surface turbulence experienced by overflow weirs.

Intuitively, it would seem that the amount of energy in the flow system would be important in determining the rate of surface renewal and the rate of turbulent mixing. The head loss across the weir should be an indicator of the surface renewed energy. In addition, for a given head loss, it intuitively seems that as unit discharge increases, the flow depth increased and the gas transfer (per unit volume) should decrease. The significance of these parameters has already been recognized by inclusion in formulations describing gas transfer in streams and rivers.

When air is entrained into the flow either on the nappe or at the plunge point into the stilling basin, the surface area available for gas transfer can increase dramatically. For example, Gulliver, et. al. (1989) estimated that entrained air due to free surface aeration increased the air-water surface area by a factor of nearly 500 compared to the unit area of surface exposed to the atmosphere. Obviously, the volume or some other indicator of entrained air is an important parameter for characterizing gas transfer at a hydraulic structure. The volume of air entrained across the free surface and at the plunge point can presently only be estimated or indirectly included in the gas transfer relationship. Once again, head loss across the weir and unit discharge are the two parameters most important to describing this process.

As the flow over a weir plunges into the tailwater, air is entrained at the plunge point. In addition to the contribution that air bubbles make to the air-water surface area, absorption of atmospheric gases from the air bubbles is enhanced because of the increased pressure that the bubbles experience as they are transported into the depth of the stilling basin. In other words, the saturation concentration of oxygen and nitrogen is higher as the bubbles move through the stilling basin.

To understand this effect, it must be recognized that the driving force in gas transfer is the "saturation deficit," which is the difference between the actual concentration of dissolved gas and the saturation concentration. The saturation concentration is dependent upon the partial pressure of the gas in air. Hence, as the bubbles are transported deep into the stilling basin, the pressure in the bubble increases (balancing the hydrostatic and dynamic pressures of its surroundings), and the saturation concentration increases. The saturation deficit thereby increases and gas absorption is enhanced. It must be noted that this effect is significant for absorption of atmospheric gases (oxygen or nitrogen) from bubble to water, but not for desorption (volatile organics or toxics) from water to bubble. Thus, the pressure history of the bubbles is an important parameter that will affect gas transfer. This effect has been identified and characterized for closed-conduit air-entrained flows by Buck, et. al. (1980) and Wilhelms. et. al. (1987).

## PARAMETERIZATION

Based on the foregoing discussion, the key parameters that could be used to describe gas transfer are: (1) head loss across the weir, (2) unit discharge, (3) volume of air entrained on the nappe of the spillway and at the plunge point into the stilling basin, (4) size of the entrained air bubbles, and (5) pressure history of bubbles transported through the stilling basin. These would all be logically included in a parametric description of $k_L$ and A in Equation 2.

Having identified the physical processes involved, the limitations of reality must be recognized: In many cases, the important parameters cannot be accurately quantified or are not included in existing data bases. Obviously, the head loss across the structure is readily available. The pressure history of bubbles moving along an "average" flow path through the stilling basin is highly dependent on discharge and structure geometry; but for some cases, the depth of tailwater may be used as an indicator of the pressure history (McDonald, et al. 1990). Some experimental work has been performed to evaluate plunge-point air entrainment (Ervine and Falvey, 1987; Mckeogh and Ervine, 1981; Mckeogh and Elsaway, 1980; Bin, 1984; Van de Sande and Smith, 1976), but the parameters required to predict air entrainment from this work are not known at hydraulic structures.

Work is currently in progress on surface aeration that occurs on the nappe of the spillway (Wilhelms and Gulliver, 1989). Of course, even if a quantitative relationship relating all these factors to transfer efficiency were established, experimental measurement of gas transfer would be required to determine a coefficient that relates these parameters to the exchange coefficient. Because of these limitations, present efforts have only included the readily available parameter of head loss in an empirical relationship for transfer efficiency at a crested overflow weir.

Butts and Evans (1978) and Rindels and Gulliver (1989) performed extensive field studies of oxygen absorption at spillways and overfalls. It is possible to identify a "generic" overflow spillway design similar to that shown in Figure 1. Analysis of data collected at seven structures (Table 1), results in Figure 2, showing a relationship between reaeration and head loss across the structure. It is highly probable that a significant amount of the variation of observed data about this relationship is a result of not specifically including the other important parameters. Until additional data are available, which include tailwater depths, air entrainment characteristics, and unit discharges, the following model of gas transfer across a low-head overflow spillway is recommended:

$$E_{20} = 1 - e^{-0.037 \Delta H} \tag{3}$$

where $E_{20}$ is the oxygen transfer efficiency at 20° C and $\Delta H$ is head loss across the structure (ft). Additional work is ongoing to include more of the important parameters in the formulation that described the exchange coefficient.

## Table 1. Structures Used in Analysis

| Structure | River | State |
|---|---|---|
| Kost Dam | Sunrise River | Minnesota |
| Elk River Dam | Elk River | Minnesota |
| Dempster Street Dam | Des Plaines River | Illinois |
| Touhy Avenue Dam | Des Plaines River | Illinois |
| Armitage Avenue Dam | Des Plaines River | Illinois |
| Hoffman Dam | Des Plaines River | Illinois |
| Devon Avenue Dam | Des Plaines River | Illinois |

## SUMMARY AND CONCLUSIONS

The physical processes that determine the amount of gas transfer at a low head overflow spillway are presented and discussed. Potential hydraulic or geometric parameters that characterize or significantly influence the physical processes are identified. The head loss across the structure and the unit discharge was suggested as an approximation for the energy per volume of flowing water, which can characterize the level of turbulent mixing and surface renewal, thereby influencing the liquid film coefficient. The amount of air entrained in the flow significantly increases the air-water surface area. The hydrostatic and dynamic pressures experienced by entrained air bubbles as they are transported into the depth of the stilling basin enhance gas absorption from the bubble into the water. Limitations on the data available for these important parameters result in a much simplified model that includes only the energy loss term. However, this simplified approximation should be acceptable when the following are considered: (1) stilling basins for the seven structures on which the approximation is based were relatively shallow (less than 5 feet), thereby minimizing any affects due to tailwater depth; and (2) the unit discharges, although not available for all the structures, appeared from photos to be on the same order (about 2 cfs). This leads to the conclusion that the proposed formulation should provide reasonably accurate predictions of gas transfer for overflow weirs with these characteristics.

$$E_{20} = 1 - e^{-0.037\Delta H}$$
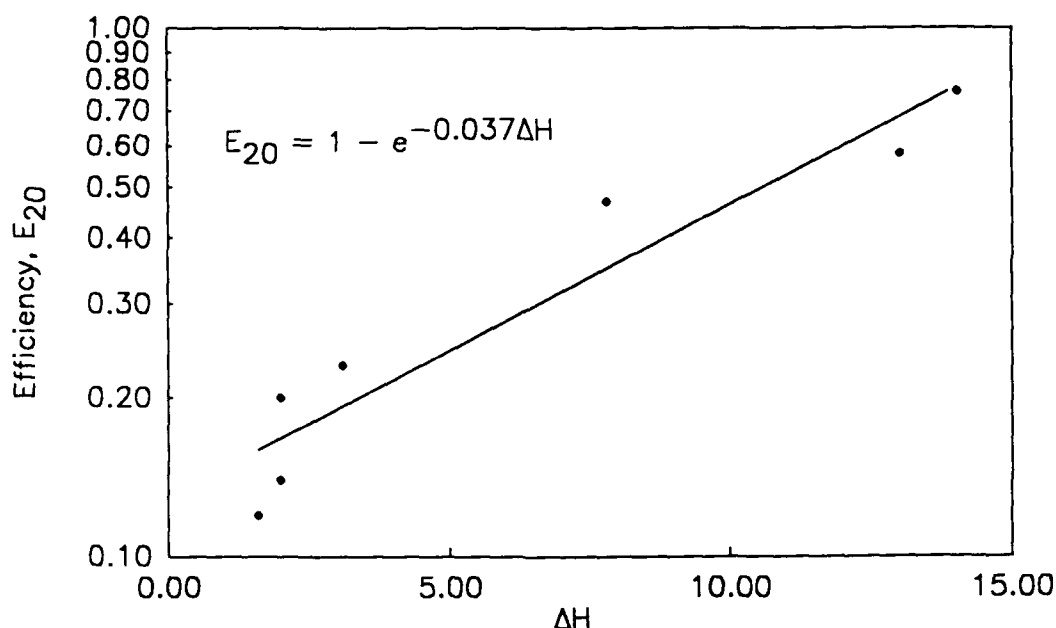
Figure 2. Efficiency Verses Head Loss for Typical Overflow Spillway

Obviously, additional theoretical and experimental work is needed to include those processes identified as being important to gas transfer. An understanding of these parameters is necessary to extend the applicability over a wider range of hydraulic and geometric conditions. Efforts in this area are continuing at the at the Waterways Experiment Station and the University of Minnesota.

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# OVERVIEW OF ENVIRONMENTAL STUDIES AT EAU GALLE RESERVOIR, WISCONSIN

by

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## INTRODUCTION

Eutrophication is frequently stimulated by the transport of sediment and associated nutrients from watershed to waterbody. While this process occurs naturally in both lakes and reservoirs, eutrophication rates in reservoirs are often greater than in natural lakes due to the relatively large watershed areas of the former (Baxter 1977; Thornton et al. 1981). Reduced water clarity, low dissolved hypolimnetic oxygen, excessive phytoplankton production and impaired recreational value are all symptomatic of the advancement of eutrophication.

Eau Galle Reservoir is a typical eutrophic impoundment in which water quality problems during the summer are associated with the excessive development of planktonic algae (Barko et al. 1984). Internal cycling of phosphorus (P) is partly facilitated in this system by wind-driven metalimnetic migrations (Gaugush 1984). Based on results of large-scale in situ enclosure experiments (Barko et al. 1986) and detailed analysis of P dynamics (James et al. 1990), it is apparent that the majority of P made available to phytoplankton during the summer may derive from sediment. These observations for Eau Galle Reservoir are consistent with those of others indicating that profundal sediments often constitute an important source of internal P loading in freshwater systems (e.g., Mortimer 1971; Riley and Prepas 1984; Nurnberg et al. 1986; Nurnberg 1987).

Like many impoundments, Eau Galle Reservoir receives P from nonpoint sources that are not readily amenable to control. In such systems, it is usually not feasible to regulate phytoplankton production by reducing external P loading. Even with reduced external loading, internal P loading can significantly delay lake recovery (e.g., Welch 1977; Larson et al. 1979, 1981; Ryding 1981). Thus, in-lake approaches to phytoplankton/water quality management need to be further developed and evaluated.

## BACKGROUND

Since 1981 we have investigated phytoplankton dynamics in relation to environmental conditions in Eau Galle Reservoir. In 1986 following several years of experimental and descriptive studies, the reservoir was treated with aluminum sulfate (alum) in an attempt to reduce the availability of internal P to the phytoplankton community. Alum application has been used successfully to reduce phytoplankton abundance in many eutrophic natural lakes by retarding P recycling from anoxic profundal sediments (e.g., Peterson et al. 1973; Soltero et al. 1981; Cooke et al. 1982; Garrison and Knauer 1983). However, to our knowledge this is the first time a flood control reservoir has been treated with alum for the specific purpose of controlling phytoplankton production.

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Alum was administered to the reservoir on 29 May 1986. Details of dose determination and application procedures are provided in Kennedy et al. (1987). Estimates of internal P loading, based on pretreatment data, indicated that the dose applied was theoretically sufficient to control sediment P release for at least 5 years. Based on total alkalinity measured at the time of treatment, the alum mass applied to the reservoir represented about one quarter of the maximum allowable dose for this system (Kennedy and Cooke 1982). Because of our decision to apply alum only to sediments subject to seasonal anoxia, i.e., at depths of 3 m and greater, about half of the surface area of the reservoir was treated. Notably, the reservoir littoral zone was entirely excluded from alum application.

The objective of this article is to provide an overview of results of environmental studies at Eau Galle Reservoir with attention to the effectiveness of recent alum treatment. For comparative purposes, results are presented from studies conducted before and after alum treatment over the period 1981 - 1988. More specific information related to the studies considered herein is reported in a WES Technical Report (Barko et al. 1990) available upon request of the authors.

## EFFECTS OF ALUM TREATMENT

The average TP loading during the summer stratified period of 1986 was substantially reduced by alum treatment, while external P loading was exceptionally high compared to other years (Figure 1). During most non-treatment years, external P loading was relatively minor during the summer stratified period. In 1987 and 1988, internal P loading was similar to pretreatment years, and accounted for the major portion of the TP load to the reservoir during summer stratification. The effectiveness of alum application in controlling internal P loading was clearly negated within the first year of treatment.

Phytoplankton biomass is presented in Figure 2 for the 8-year study period. Average summer biomass ranged between 10 and 20 g/m3 fresh weight during all years except 1985, when it was much greater at 51 g/m3 following reservoir drawdown the previous winter. Alum treatment in 1986 did not affect phytoplankton biomass that year or in either of the two following years (except relative to 1985). Excluding 1985, there were essentially no differences in average summer biomass or biomass maxima over the entire eight-year study period.

## FACTORS AFFECTING TREATMENT EFFECTIVENESS

Rainfall over the watershed was particularly heavy in July of both 1986 and 1987. Major freshets caused peaks in external TP loading from the Eau Galle River throughout the summer of 1986 and also during the late summer of 1987. Elevated external TP loading, due to unusually high precipitation, probably played an important role in mitigating the effectiveness of alum treatment in controlling phytoplankton production in 1986. Had alum been applied to the reservoir during dry years (e.g., 1982 and 1988), when internal TP loading accounted for most of the TP mass in the reservoir, control over phytoplankton production might have been more successful.
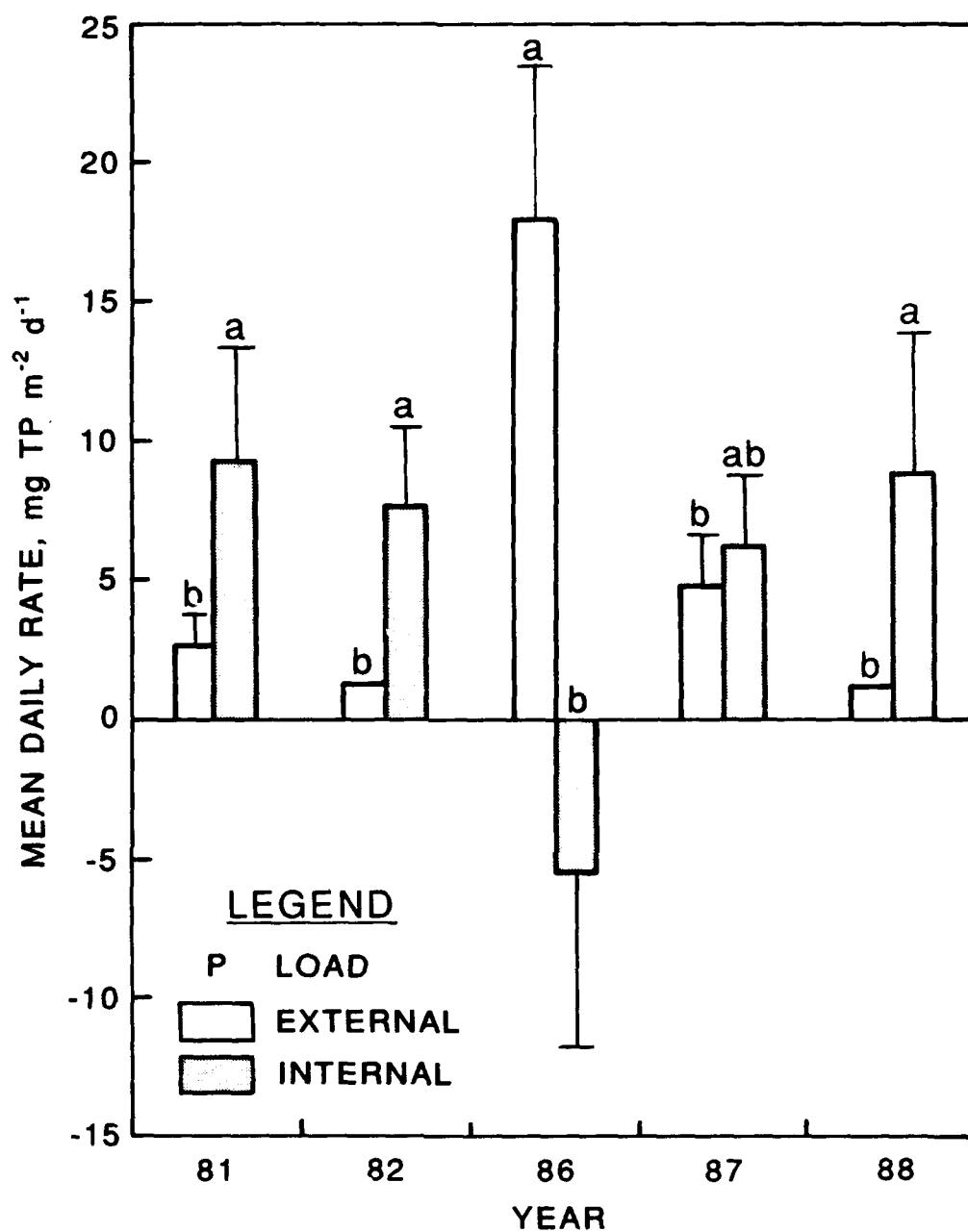
Fig. 1. Mean daily external and internal TP loadings to Eau Galle Reservoir during mid-summer periods (June - August) for 1981, 1982, and 1986 - 1988. Different letters associated with individual variables indicate significant differences at the 5% level or less, based on Duncan's Multiple Range Analysis. Vertical lines represent one standard error of mean values. Alum was applied to the reservoir in 1986. The negative internal loading rate in 1986 reflects sedimentation.
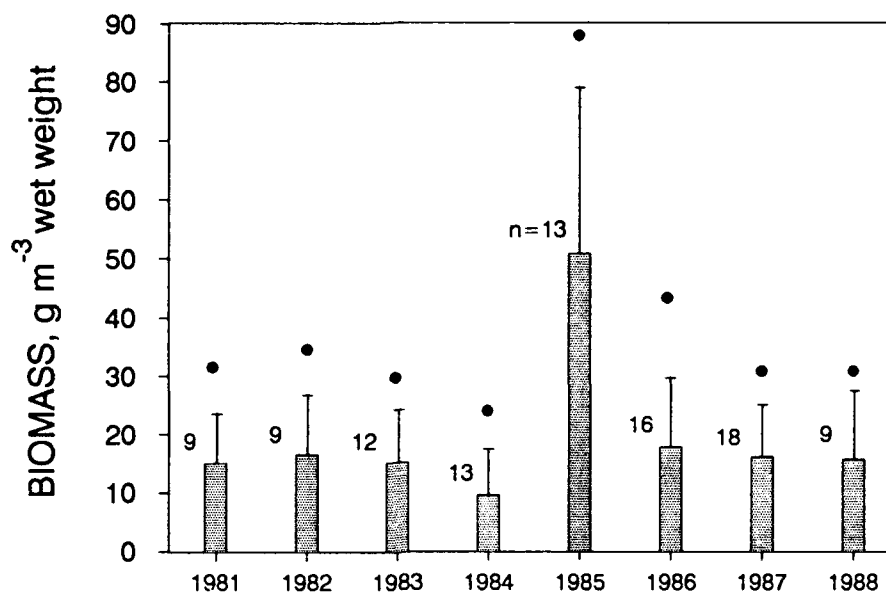
Fig. 2. Mean summer (June-September) phytoplankton biomass with 1
standard deviation, n, and the maximum summer biomass
(denoted by the filled circles).

Burial of alum by sediment transported during periods of high inflow in
1986 may have been another important factor in shortening the effectiveness
of alum treatment in this system. Eau Galle Reservoir, since its
construction about 20 years ago, has experienced an average sedimentation
rate of 2-3 cm./y in the deep basin, based on detailed examination of sediment
cores (James and Barko 1990). The sedimentation rate during 1986 was
probably considerably greater than this average rate. As in our
investigation of Eau Galle Reservoir, Foy (1985) suggested that sedimentation
was responsible for the rapid recovery of hypolimnetic P concentrations to
pretreatment levels following alum treatment in White Lough. In contrast,
effective control of sediment P release in natural lakes with lower
sedimentation rates has ranged from 3 to 8 years (e.g., Born 1979; Cooke et
al. 1982; Garrison and Knauer 1983).

Groundwater discharge may provide another internal source of P to the
epilimnion of Eau Galle Reservoir. Indeed, hydraulic discharge from this
reservoir has exceeded water income throughout our studies. This imbalance
was particularly apparent during the relatively dry summers of 1982 and 1988.
It has been suggested that groundwater flow through P-rich sediments may be
an important internal P-loading mechanism in West Twin and Dollar lakes in
Ohio (Cooke et al. 1978, 1982).

Recent investigations have indicated that littoral sediments may release
substantial amounts of P, particularly at high pH (Twinch and Peters 1984;
Drake and Heaney 1987). Using sediments from the littoral zone of Eau Galle
Reservoir, we have measured P release rates approaching 5 mg/m2/day at pH 9.0
in a controlled environmental chamber (James and Barko unpublished data).
These rates are comparable to calculated internal P loading rates from
profundal sediments under anaerobic conditions during the summer stratified
period. Since the littoral zone was not treated with alum, it is possible
that littoral sediments may have contributed P, in addition to external
inputs, to the phytoplankton community following alum treatment.

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## SUMMARY AND CONCLUSIONS

Although internal P loading predominates during the summer stratified period in Eau Galle Reservoir, unpredictable episodic freshets frequently result in substantial external P inputs to the epilimnion. Both internal and external sources of P are important in sustaining high biomass of phytoplankton in this system.

Alum treatment clearly and significantly reduced internal P loading from profundal sediments in 1986, but had no effect on phytoplankton production. Abundant P is available to the phytoplankton community of Eau Galle Reservoir from multiple sources. Major sources of P input include the river, profundal sediments, littoral sediments, and possibly groundwater discharge.

Internal loading of P in Eau Galle Reservoir probably cannot be controlled over the long term by alum application alone. Thus, other means of reducing availability of P to the phytoplankton community need to be developed. As with many other eutrophic reservoirs, control of nutrient inputs from external sources is difficult to achieve in Eau Galle Reservoir. Control of internal sources of P loading to this reservoir appears to be equally difficult to achieve.

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# EFFECTS OF ALUM TREATMENT ON PHOSPHORUS DYNAMICS
# IN A NORTH-TEMPERATE RESERVOIR

## BY

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## INTRODUCTION

Ferric aluminum sulfate and aluminum sulfate (alum) treatments have been widely used in lakes to curtail internal P loading from the sediment, and thereby reduce phytoplankton biomass (Peterson et al 1973; Soltero et al 1981; Cooke et al 1982; Garrison and Knauer 1983; Foy 1985). Most natural lakes receive their P income primarily from internal sources during the summer. However, reservoirs and lakes associated with large watersheds may also receive substantial external P inputs from inflowing tributaries, making them potentially more difficult to manage. Herein, we examine the effects of a hypolimnetic alum treatment on sediment P availability in Eau Galle Reservoir. Results reported here span pretreatment (1981-1982) and post-treatment (1986-1988) years.

## METHODS

Alum (11.3 g Al m-2; 4.5 mg Al L-1) was administered to the reservoir on 29 May 1986. Details of dosage and application procedures are provided in Kennedy et al (1987). The treatment area was confined to depths of 3 m and greater, since sediments at these depths experience anoxia during summer stratification. Here, we assumed that only anoxic sediments contributed to the internal P load.

Lake water samples for chemical analyses were collected weekly or biweekly at 1-m intervals (or closer) throughout the summer months in 1981, 1982, 1986, 1987, and 1988 at a deep (9 m), centrally-located station. River water samples for chemical analyses were collected weekly (or more frequently) at fixed stations located on the Eau Galle River, Lohn Creek, Lousey Creek, French Creek, and the reservoir tailwater. Detailed sampling procedures are provided in Barko et al (1990). Total phosphorus (TP) was determined colorimetrically on a Technicon Auto-Analyzer II following persulfate oxidation (APHA 1985). SRP was determined following filtration (0.45 um) using the ascorbic acid method on either a Technicon Auto-Analyzer II or a spectrophotometer (APHA 1985). Details on the determination of external TP loadings are provided in Barko et al (1990). Internal TP loading was determined

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according to the following mass balance equation:  Change in TP
Mass = (External TP Load - TP Outflow) + Internal TP Load.
Statistical analyses were performed using the Statistical Analysis
System (SAS 1985).


## RESULTS


Eau Galle Reservoir commonly exhibited elevated concentrations
of SRP in the hypolimnion before alum treatment, as in 1982
(Fig. 1).  Shortly after the occurrence of anoxic conditions,
concentrations of SRP increased above the sediment surface,
indicating P release from anoxic sediment.  Steep vertical
gradients in SRP were observed and the concentration exceeded 1.00
mg P L-1 within the bottom meter from June through September of
that year.  Alum treatment resulted in low SRP throughout the water
column from June until late July, 1986.  Steep vertical gradients
in SRP again developed below the 6-m depth during late August,
1986.  However, absolute SRP concentrations in the hypolimnion were
much lower in 1986 than in 1982.  Slight increases in SRP at
intermediate depths (4 to 6 m) in mid-August  1986, coincided with
high external TP loading.  In 1987 and 1988, hypolimnetic SRP
increases were very similar to those observed in 1982.  Periods of
major external TP loading caused increases in SRP throughout the
water column in September  1986, August  1987, and September  1988.

During the stratified periods of 1981 and 1982, internal TP
loading frequently exceeded 10 mg m-2 d-1 (Fig. 2), and was 3 to 6
times greater than external TP loading (Table 1).  Alum treatment
completely negated internal TP loading from June through mid-July
1986.  During that period, negative internal TP loading rates
reflect losses through sedimentation and/or discharge.  Positive
internal TP loading resumed in late July and August of 1986.  In
1987 and 1988, internal TP loading was similar to pretreatment
years (1981 and 1982) and accounted for 58-85% of the total TP load
to the lake during summer stratification (Table 1).

Before alum treatment, increases in epilimnetic TP mass
usually corresponded with periods of elevated internal TP loading.
Similarly, during the 1988 drought (after alum treatment),
epilimnetic TP mass exhibited increases during periods of elevated
internal TP loading.  In contrast, external TP loading appears to
have been the major contributor to TP mass in the epilimnion during
1986.  In 1987, both internal and external TP sources contributed
to TP mass in the epilimnion.

DISCUSSION

Before alum treatment, increases in epilimnetic TP mass were common in Eau Galle Reservoir during periods of minimal external TP loading, indicating that internal sources of TP were responsible for these increases. The occurrence of SRP increases above the profundal sediment surface following the onset of anoxia, and the accumulation of SRP in the anoxic hypolimnion, both indicated that P release from sediments was an important mechanism of internal P loading. Investigations conducted in a variety of lakes have shown that P released from anoxic sediments can account for a substantial portion of the internal TP load and TP mass increase in the water column (Nurnberg 1984, 1987; Riley and Prepas 1984).

Increases in epilimnetic ₁P mass have been observed in this reservoir following periods of decreased thermal stability, when wind-driven mixing results in a slight downward depression of the thermocline (Gaugush 1984). Thus, entrainment of P from hypolimnetic water or from metalimnetic shelf sediments appears to be an important mechanism for vertical P transport to the epilimnion. Biological uptake and therefore transport of P to the epilimnion by vertically migrating phytoplankton (Salonen et al 1984) may also be an important factor. The vertically migrating dinoflagellate, Ceratium hirundinella, usually dominates the phytoplankton assemblage in this reservoir during the summer (Barko et al 1984; Barko et al 1990) and has been shown to migrate into the metalimnion at night (Taylor et al 1988).

Alum treatment resulted in a substantial reduction in hypolimnetic TP and internal TP loading during 1986. However, the abnormally high frequency of major external TP loading events during that year negated the effectiveness of alum treatment in reducing epilimnetic TP mass. Epilimnetic TP mass in 1986 remained essentially unchanged from pretreatment years.

The dose of alum applied to Eau Galle Reservoir was theoretically sufficient to control sediment P release for 5 y (Kennedy et al 1987). Nevertheless, the effectiveness of alum treatment in controlling internal TP loading was greatly diminished one year after treatment. Several possible factors may have contributed to lack of success in treating this reservoir with alum. These include burial and inactivation of alum by sedimentation, P release from untreated sediment, and P inputs from ungauged hydraulic sources.

The reservoir has an average sedimentation rate of 2-3 cm y-1 in the deep basin (James and Barko 1990). Thus, sedimentation was potentially a most important factor in reducing the effectiveness of alum treatment. Since we did not treat the shallow water sediments of this reservoir, it is also possible that epilimnetic TP mass was influenced during periods of low external TP loading by

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internal TP loading from these areas. Hydraulic discharge has exceeded water income throughout our studies, suggesting that ungauged groundwater source inputs of TP provided another internal source of P to this reservoir.

Although internal P loading predominates during the summer stratified period in Eau Galle Reservoir, episodic freshets frequently result in substantial external P inputs to the epilimnion. Both external and internal P sources are important in sustaining high concentrations of TP, as well as high phytoplankton biomass in this reservoir (Barko et al This Proceeding). Internal P loading appears to be much more important than external P loading in lakes with small watersheds and minimal tributary inputs (e.g. Cooke et al 1977; Larson et al 1981; Stauffer 1987). In comparison with these lakes, reservoirs such as Eau Galle Reservoir, and natural lakes as well that receive substantial inputs from tributaries draining large watersheds, may experience generally greater external loads, making them difficult to manage.

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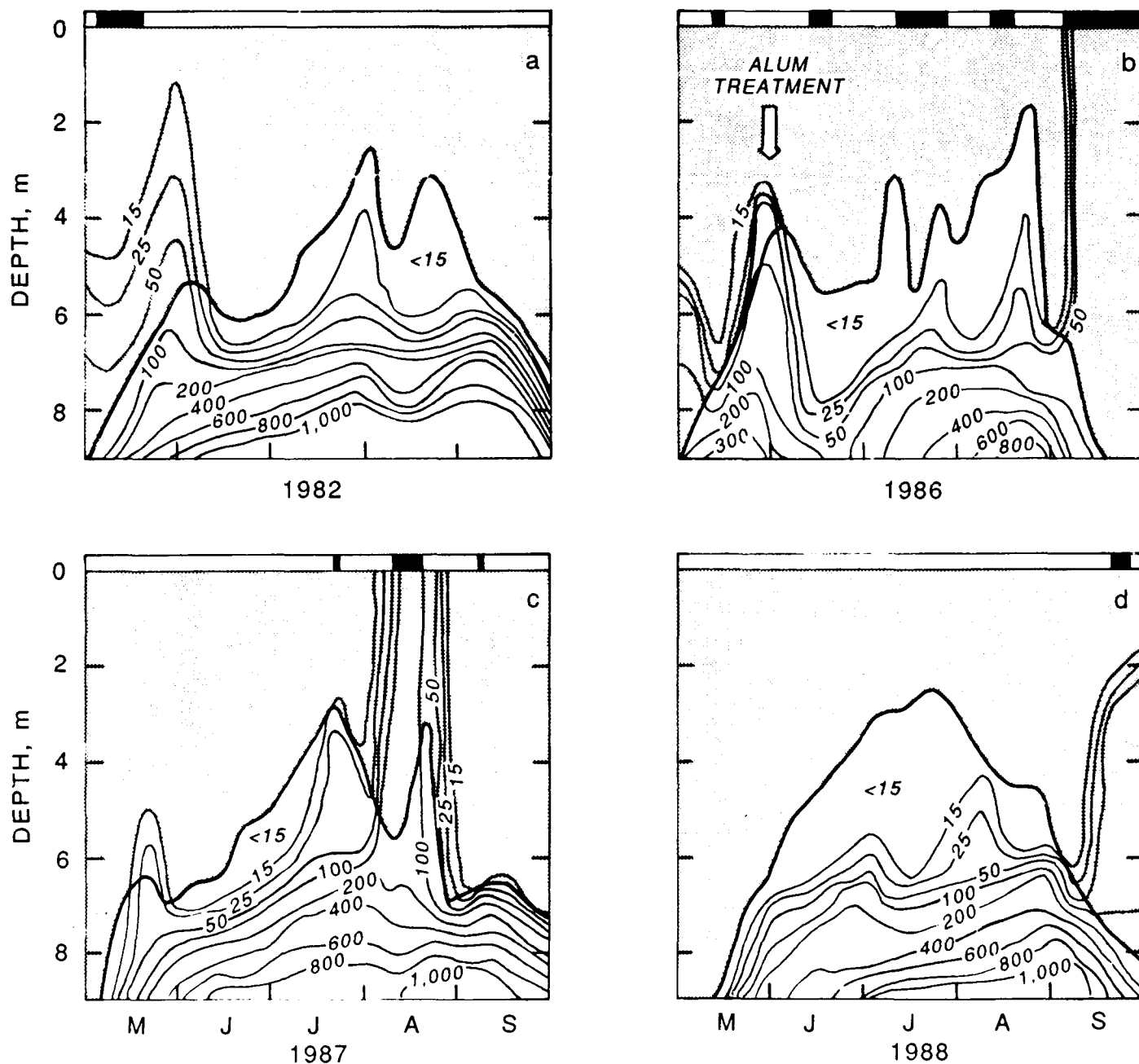
Fig. 1. Seasonal and depth-related variations in soluble reactive
phosphorus (ug/L). Measurements were made at close depth
intervals (25 cm) in a) 1982, b) 1986, c) 1987, and d) 1988
for the period May through September. Shaded area indicates
the zone where dissolved oxygen was > 0.5 mg  .-1. Black
areas above figure panels indicate periods during which
external TP loading exceeded 25 kg TP d-1.

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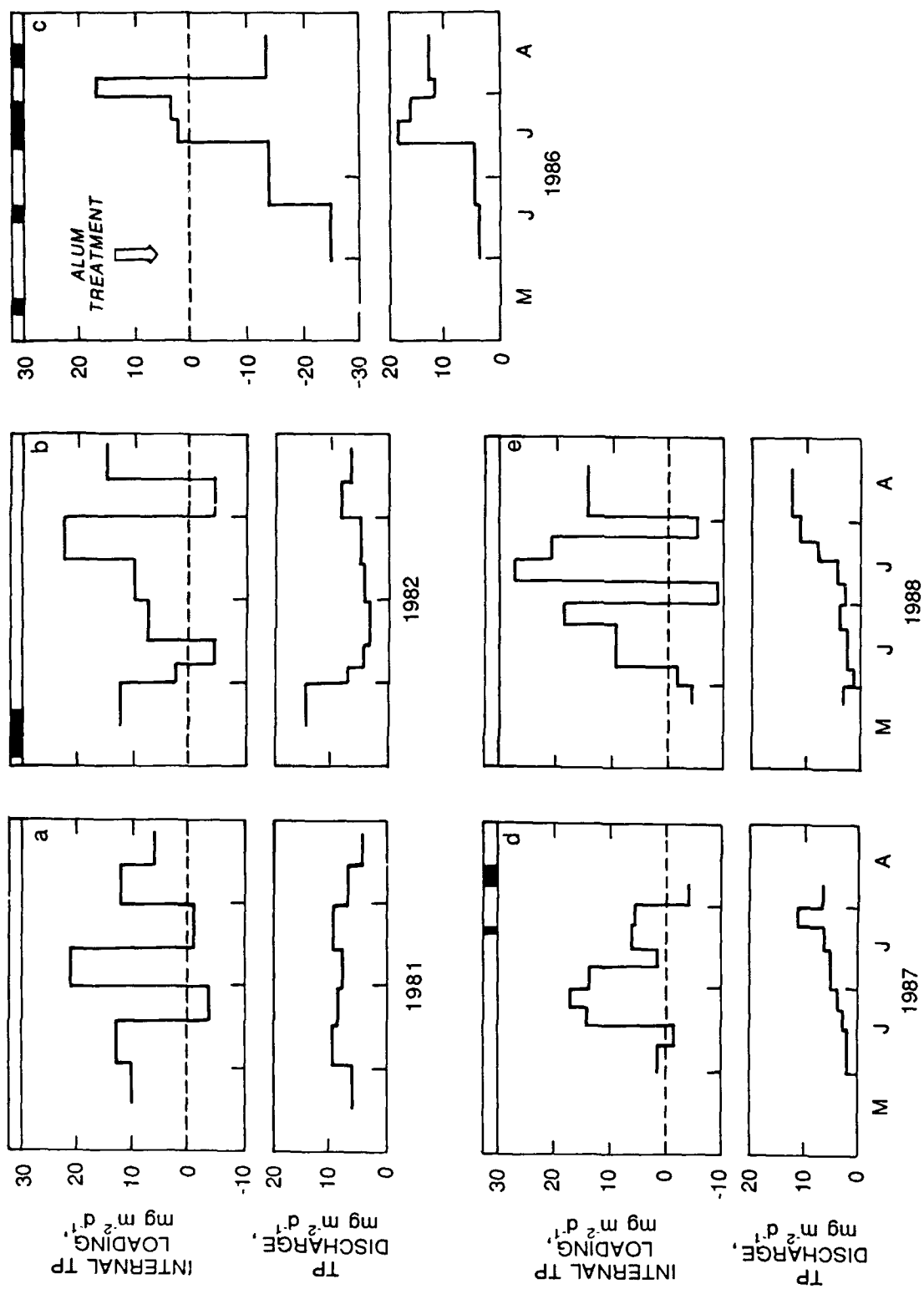
Fig. 2. Seasonal variations in internal TP loading (upper panels) and TP discharge (lower panels) in a) 1981, b) 1982, c) 1986, d) 1987, and e) 1988. Dashed lines indicate an internal TP loading rate of zero. Black areas above figure panels indicate periods during which external TP loading exceeded 25 kg TP d-1.

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Table 1. Mean (standard error given in parentheses) daily internal and external TP loadings to Eau Galle Reservoir during mid-summer periods (June-August) for 1981, 1982, 1986-1988. ANOVA column summarizes results of Duncan's Multiple Range Analysis. Different letters associated with individual variables indicate significant differences at the 5 percent level or less. Alum was applied to the reservoir in 1986.

| Year | Internal TP Loading (mg m-2 d-1) | n | ANOVA | External TP Loading (mg m-2 d-1) | n | ANOVA |
|------|------|---|-------|------|----|-------|
| 1981 | 9.31 (4.13) | 7 | a | 2.76 (1.14) | 8 | b |
| 1982 | 7.84 (2.87) | 8 | a | 1.28 (0.01) | 92 | b |
| 1986 | -5.58 (6.21) | 6 | b | 17.01 (5.67) | 85 | a |
| 1987 | 6.25 (2.60) | 9 | ab | 4.89 (1.96) | 65 | b |
| 1988 | 8.97 (5.01) | 8 | a | 1.29 (0.03) | 85 | b |

# RETENTION OF STANDING TIMBER FOR
# FISH AND WILDLIFE HABITAT IN RESERVOIRS

by

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and
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## ABSTRACT

The question of whether to maintain standing timber in an Oregon multipurpose reservoir constructed by the U.S. Army Corps of Engineers was evaluated by project managers and debated by the community in which the reservoir was located. Although the practice has been accepted in other parts of the country, it was a relatively new idea for the Corps in the Pacific Northwest. Issues that were considered included impact on fish and wildlife habitat, recreation, boater safety, water quality, and debris removal. Evaluation based on experiences in other reservoirs and cost-benefit analysis indicated that fish and wildlife habitat would be enhanced; boating, recreation, and water quality would not be adversely affected; and the community would benefit financially over the 50-year life of the project, even taking into account the cost of debris removal. Community objections were aired and resolved, and the U.S. Army Corps of Engineers directed that 245 acres of standing timber in the reservoir basin be retained.

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COASTAL AND ESTUARINE SESSION

# EVOLUTION OF DREDGED MATERIAL DISPOSAL MANAGEMENT
# IN SAN FRANCISCO BAY REGION

by

T. H. Wakeman, T. J. Chase and D. E. Roberts[1]

## INTRODUCTION

Navigation channel maintenance and improvements are essential to the nation's ability to effectively compete in international import/export markets. The San Francisco Bay and estuary act as a critical thoroughfare for the nation's increasing role in Pacific Rim Trade with its numerous ports and intermodal links. Furthermore, the Bay and estuary, the largest coastal embayment on the Pacific coast of the United States, are also a significant habitat for anadromous and marine fish and other species that have a high economic and resource value to the region. Over the last twenty years, the competing needs of these different beneficial uses have become increasingly controversial.

As of 1983, the San Francisco Bay Area was the fifth largest export manufacturing center in the United States with export-related employment of over 68,000 and a dollar value of close to 7 billion (Skinkle, 1989). In 1980, trade with the Pacific Rim nations (Japan, Korea, Taiwan, Australia and other countries in the Far East) accounted for one-quarter of the nation's imports/exports -- today the share is over one-third and rising (Skinkle, 1989).

Maintenance of existing channels and design of channel improvements requires consideration of navigational safety, economic impacts and environmental effects. The U.S. Army Corps of Engineers, San Francisco District, currently dredges and disposes over 4 million cubic yards (mcy) of sediment annually from both deep- and shallow- draft Federal navigation channels in the San Francisco Bay region; another 3 mcy of sediment are dredged and disposed under Corps-issued permits. In addition, a number of channel improvement projects have been authorized by Congress requiring the dredging and disposal of more than 19 mcy.

Historically, disposal of the majority of this dredged material has been at open-water disposal sites in the Bay. However, disposal in open water causes increased turbidity and disruption of benthic habitats. Several Federal and State agencies have voiced increasing resistance to this practice. In addition, the primary disposal site in the Bay has become limited as to the quantity of material it can accept. Thus, the San Francisco District has responded with a comprehensive program to develop a disposal management strategy that balances both economic and environmental perspectives.

## SEDIMENT DYNAMICS

The San Francisco Bay estuary is a drowned river valley through which passes the drainage of the great Central Basin of California. The Bay has an area of 396 square miles (sq. mi.) at mean lower low water and 460 sq. mi. at mean higher high water (USACE, 1977a). Extensive intertidal mudflats, encompassing an area of 64 sq. mi., are exposed at lower low water. The Bay is generally shallow with two-thirds of the area less than 18 feet (ft.) deep, and only 20 percent of the Bay is greater than 29.5 ft. deep (USACE, 1977a).

An estuary is both a sink and a holding area for fluvial sediment in transit to the ocean from soil erosion in the estuary's drainage system. The estimated average annual sediment inflow to San Francisco Bay ranges from approximately 8 to 10.5 mcy (USACE, 1979). Smith (1965) used

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hydrographic surveys of the Bay taken between 1855 and 1956 to show an annual sediment accumulation from Suisun Bay through North San Francisco Bay of 5.5 mcy. Most of this accumulated material resides in the northern and central portions of San Francisco Bay. South San Francisco Bay actually shows a net loss of 0.9 mcy per year (Krone, 1979). Sediment outflow through the Golden Gate is generally estimated to range between 3.9 to 5.2 mcy (USACE, 1979).

Although millions of cubic yards of sediment move into and out of the estuary, these volumes are really only a small part of the total sediment cycling in the system. Within the sediment regime of the Bay, the major source of suspended sediments is resuspension of previously deposited material by tidally dominated currents and, especially in the shallower areas of the Bay, by waves. These waves can be induced by prevailing westerly winds in the summer or strong Pacific storms in the winter. The quantity of sediment annually resuspended in the shallow areas by wind waves and wind driven currents has been estimated to be between 160 to 170 mcy (USACE, 1979).

The net movement of sediment in the estuary is downstream. This material ultimately either moves out the Golden Gate to the Pacific Ocean or deposits in areas where there is insufficient energy for further resuspension and transport. The best example of such an area is anywhere dredging must occur to maintain a prescribed depth. The artificially deepened navigation channels, ports and marinas of the Bay often act as sedimentation basins that capture the circulating suspended particles as the water velocity slows. Many of these areas are adjacent to shallow mudflats and receive sediments freshly resuspended by wind-wave action. It is interesting to note that the annual dredging activity for the Bay closely mirrors the annual inflow of sediments to the system.

## DISPOSAL HISTORY

The construction and maintenance of navigation channels in the San Francisco Bay began with the passage of the River and Harbor Act of 1868. With respect to this Bay, the purpose of the legislation was to provide a deepwater port in an otherwise naturally shallow embayment to handle the inflow of supplies and equipment for the miners in the Sierra Nevada foothills. Subsequently, as the population in the region grew, so did the need for new channels, ports and marinas. During the period between the 1950s to the 1970s, there was an annual requirement for approximately 10 mcy of maintenance dredging. More recently, this volume may be closer to 8 mcy (Beeman, 1988).

In addition to the on-going maintenance requirements for the channels and harbors, there is also a continuing demand for channel improvements to support ever changing commercial and military needs. Presently, future new work for Congressionally authorized civil works projects (Oakland and Richmond harbors and John F. Baldwin ship channel, Phase III) and U.S. Navy construction is estimated to be approximately 19 mcy.

Prior to 1972, material dredged from San Francisco Bay was disposed at eleven known aquatic disposal sites within the Bay as well as several undocumented sites. Physical model studies were conducted to assess suspended sediment movement following disposal. These studies were performed during 1963 for the Corps' Report of Survey on San Francisco Bay and Tributaries, California (USACE, 1967). Based on the model tests, it was determined that disposal areas located closer to the Golden Gate would facilitate movement of material to the ocean. The Committee on Tidal Hydraulics reviewed these data in 1965 and suggested that the District may be redredging a significant portion of the material remaining in the Bay following open water discharge (USACE, 1965).

In 1972, the District responded to the comments from the Committee on Tidal Hydraulics and the State by limiting dredged material disposal activities to five sites within the Bay. This action was taken to decrease the amount of dredged material potentially being redredged throughout the Bay and to provide for better regulatory control. In addition, in 1972, the San Francisco Bay Regional Water Quality Control Board (RWQCB) adopted the U.S. Environmental Protection Agency's (EPA) Jensen Criteria. (Prior to 1972, there were no recognized criteria for evaluating sediment quality. In 1972, the Jensen Criteria for the Great Lakes were published by the EPA.) The criteria provided a definition of polluted versus nonpolluted sediment based on concentrations of mercury, cadmium, lead, zinc and

oil/grease. This bulk sediment characterization approach was tailored to the region to assess dredged material quality. The usage of the established Bay sites and ocean sites (a site near the San Francisco Bar channel and an interim EPA designated 100-fathom site) was evaluated based on results of the Jensen bulk sediment test. Sediments that were classified as unpolluted would be acceptable for disposal at any Bay disposal site or, given the proper particle size distribution, at the Bar site. All sediments that were classified as polluted by the Jensen criteria required disposal either on land or at the 100-fathom site.

As research on dredged material disposal evolved during the early 1970's, it became apparent that the Jensen Criteria were not useful in either predicting water column effects or providing biologically relevant data. With the development of the elutriate test in 1975 and the promulgation of the 1977 Ocean Dumping Regulations (with their accompanying implementation manual), a new set of tests was available to replace the sediment quality criteria. These new procedures became the methodologies for assessing dredged material acceptability for in-Bay and ocean disposal.

In 1978, the San Francisco District specified the aforementioned procedures for evaluating dredged material disposal in San Francisco Bay in its Public Notice (PN) No. 78-1. Furthermore, the District used this PN to establish its policy of utilizing only dispersive sites and formally designated the three present in-Bay disposal sites. These sites were selected to: (1) locate all disposal in areas of high current energy to enhance dispersion of the material; (2) locate the sites as close to the ocean as possible to facilitate transport of the material to the ocean; and (3) localize potential disposal impacts.

The RWQCB adopted the District's PN 78-1 regional guidelines in its Resolution No. 80-10, dated August 1980. These three sites are located at Carquinez Strait, San Pablo Bay and near Alcatraz Island. Presently, the Carquinez disposal site receives approximately 1.5 mcy of dredged material annually from the Corps maintenance of the Mare Island Channel as well as minor permit contributions. The San Pablo Bay disposal site receives approximately 500,000 cubic yards of dredged material annually from maintenance dredging activities at Petaluma River channel and a portion of the maintenance work from the Pinole Shoal channel. In addition, small quantities of material from permits are periodically disposed at the site. The Alcatraz disposal site is the primary disposal site for the region, receiving maintenance and new work materials from projects located in both Central and South Bay. Between 1984 and 1988, based on dredging records during this period, the disposal site received approximately 4 mcy annually.

The approach of disposing of the majority of dredged material at Alcatraz was justified as being economically and environmentally sound for several years. However, in November of 1982 mounding at the Alcatraz disposal site was detected. Initially efforts to recover the dispersive characteristics of the site were attempted including removal of 30 tons of construction debris. After directing on-going disposal away from the mound, it was noted that natural currents failed, over two winter seasons, to reduce the mound. Accumulation continued after dredging had leveled the peak of the mound. As field investigations progressed, it became apparent that the site could not accept dredged material in an unlimited fashion. The fact that Alcatraz could no longer be considered a fully dispersive site led to questioning of the biological impact of the accumulated sediments.

Bioassays were included in the water quality evaluation procedures for dredged material disposal as specified in PN No. 78-1, but were only required if the elutriate test results exceeded known standards or statistical comparisons of disposal site concentrations. Since the elutriate test results rarely exceeded the standards, most disposal activities occurred within Bay waters without biological testing.

In 1987, the District, EPA and the Regional Board determined that water column testing (elutriate) was inadequate to protect the water quality of the Bay. They jointly issued, in draft form, Public Notice 87-1. Although never finalized, the public notice provided new regional procedures for evaluating the discharges of dredged material within San Francisco Bay that are presently being followed by the District and RWQCB. According to the procedures described in PN 87-1, a tiered testing approach including chemical analyses of sediment replaced the chemical analyses of the elutriate of the sediment sample as required by PN 78-1.

As previously mentioned, sediments that were determined to be unsuitable for Bay disposal had historically been taken to the ocean. Before 1980, the San Francisco District had two interim designated ocean disposal sites: the 100-fathom site, about 28 nautical miles (nmi) from the Golden Gate and the San Francisco Bar Channel site, located about 5 nmi from the Golden Gate near the San Francisco Main Ship Channel. The 100-fathom site was used for material from Oakland Harbor in 1975 that was considered too polluted for Bay disposal. The site now lies within the Point Reyes-Farallon Islands Marine Sanctuary, which was designated by the National Oceanic and Atmospheric Administration (NOAA) in 1980. Following this decision by NOAA, the site was removed from the final designation process by EPA in 1983. At this time, the only authorized disposal site offshore San Francisco is the San Francisco Bar Channel site; it received final designation in September 1985. Currently, the use of the site is designated for only coarse-grained material (defined as 95 percent retained on a No. 200 standard sieve).

With the loss of the 100-fathom ocean disposal site in 1980, the alternative of ocean disposal for dredged material from San Francisco Bay was effectively eliminated. After initial coordination with concerned agencies, it was determined that there was no consensus on appropriate areas to survey for a new site. In order to proceed with the designation process for a new site, the District identified one nearshore area within a short distance from the Gate and one area along the 100-fathom contour outside of the Marine Sanctuary to be surveyed.

Ocean site surveys were conducted in 1983. Following the review of these data, it was concluded that neither site would probably be acceptable. In 1985, further assessment of five additional alternate sites was determined necessary to adequately address alternative candidate locations under the NEPA process. The five additional areas were surveyed in 1985 and 1986. These additional sites were located in areas where as many known resources and uses could be avoided as possible. Furthermore, the sites were selected to be approximately the same distance from the Golden Gate as the interim designated 100-fathom site that was eliminated from EPA's list of designated sites.

Bathymetric surveys at the five areas were reviewed and two sites were eliminated from further consideration due to irregular bottom contours. No further studies were performed at these two sites. Additional detailed data collection related to chemical, biological, and physical parameters continued to be collected at the remaining three locations.

Data from these sites as well as two additional sites added in 1987 were presented in a Supplemental Environmental Impact Statement (SEIS) for Oakland Harbor Project. The document recommended disposal of the material from the project at Site B1. Following review of the ocean disposal alternatives, a Corps/EPA panel recommended disposal at Site B1B, a new site situated between B1 and another surveyed site. This recommendation was accepted in the Division Engineer's Record of Decision signed on 4 May 1988. Subsequently, a law suit prompted by angry fishermen was filed, and a State Court injunction stopped the Port of Oakland from continuing to dispose of dredged material at this site. Thus, a usable ocean site still eludes the region.

DISPOSAL POLICY

The District's disposal policies have been constantly evolving since the early 1970's when disposal of dredged material in the region first began to be questioned. For several years (1970-1977). the District depended on historic precedent while it conducted studies to determine the most suitable policy position. The policy, as instituted in PN 78-1, was based on regional acceptance of dispersive disposal sites in San Francisco Bay. This perspective was developed from historic observation, physical model studies and the results of the District's $3.5 million Dredge Disposal Study (USACE, 1977b). However, the utility of the dispersive disposal policy was weakened when mounding at Alcatraz, the primary disposal site, was noted in 1982. Furthermore, it was obvious that the mounding of discharged sediments would critically limit disposal options within San Francisco Bay.

In recognition of the potential severity of the shoaling problem on continued operations, the San Francisco District implemented an interim disposal policy to minimize material deposition at the existing

disposal sites in 1984. This policy used management measures that included a slurry requirement to enhance dispersion. An interim policy decision structure implies that some kind of new policy is being developed. Indeed, a new effort was initiated by the District because of the renewed need to fully identify all feasible disposal solutions and to develop a long-term, implementable management plan for their utilization. This effort was identified as the Dredged Material Disposal Management Program.

## DISPOSAL MANAGEMENT PROGRAM

The initial statement of work for the Dredged Material Disposal Management Program (DMP) was presented in a plan dated 12 June 1985. At that time, the plan stated that a final policy formulation document would be prepared in 1989. The policy document would represent the results of the previous five years of DMP technical studies, including the completion of ocean disposal site designation activities. Unfortunately, plans are not always achieved as initially laid out. In the case of the DMP, many events transpired over the next five years that negated the possibility of establishing a final dredged material disposal policy for the region in 1989. These events generated new issues and public interest in the disposal of dredged material not envisioned in 1985.

The DMP must now address these issues and respond to public concern in order to successfully complete its task. The program was reconfigured in 1989 to study the new concerns and to fit the management plan development into the Long-Term Management Strategy (LTMS) approach (Francinques and Mathis, 1989). The development of a LTMS is consistent with U.S. Army Corps of Engineers dredging regulations (33 CFR Part 337.9).

The timeframe for the LTMS is twenty-five (25) years. The scope of the program encompasses all present and future activities related to the relocation of dredged material in the San Francisco Bay region. However, the completion of a LTMS will require additional time (beyond the 1989 timeframe set in 1985) and additional studies (beyond those completed thus far) in order to reach the goal of a regional disposal management plan. Furthermore, there will be a continuing requirement because under the LTMS approach, review and updating of the plan would continue on a routine basis.

During the initial DMP organization, five major work units were set up: (1) Alcatraz Investigations; (2) North Bay Site Monitoring; (3) Sediment Transport Modeling; (4) Other Disposal Options; and (5) Ocean Disposal Site Designation. As suggested by the work unit categories, most of the emphasis at that time was on the existing sites and the ocean alternative. There were investigations to characterize and monitor use of Alcatraz and the North Bay sites. The modeling work unit was to provide assistance with management of material discharged at these sites. Several new areas within the Bay were also being considered to provide additional in-Bay options. Concurrently, an on-going study of alternative site(s) in the ocean to replace the interim designated 100-fathom site was accelerated

In the early years, the main concern facing the DMP was operational in character (i e , where to put sediments). With the accumulation of material at Alcatraz and the implementation of the slurry requirement, new issues arose during the mid-1980's. These issues were, and continue to be, environmental in character. The accumulation of material at the site has caused concern because only water column effects had been examined for many years. The deposited material may be acting as a source of contaminants to benthic species burrowing through the substrate or feeding on organisms that inhabit it. Because material is dredged from ports and harbors, its contaminant burden may be different than the sediments in the open Bay. Concerns have been voiced that Alcatraz's chemical makeup has been degraded by contaminants associated with backharbor sediments. Furthermore, the mound at Alcatraz has changed the habitat as well as buried the indigenous species that had previously occupied that location.

Another issue that has been raised, separate from the accumulation problem, is the impact of turbidity in the Central Bay on fisheries. That issue is an outgrowth of the slurry requirement. Fishermen have alleged that since the slurry requirement was implemented at Alcatraz, Central Bay fisheries have declined. It has been suggested that the problem is caused by both physical and chemical impacts at the site. Physical effects include simple avoidance of the disposal plume as well as gill abrasion or other

sub-lethal irritations caused by solids in the water column. Chemical effects proposed as causes of the fisheries decline include release of ammonia or hydrogen sulfide as well as direct toxicity from contaminated sediments.

These environmental concerns were discussed at several public hearings held in early 1989 -- RWQCB (February 15 and March 15, 1989), BCDC (March 16, 1989), EPA/Corps (April 11, 1989) and State Lands Commission (April 12, 1989). The discussions have not only involved Alcatraz but also stipulated concerns for all in-Bay disposal operations as well as the ocean and upland options.

When the District established the DMP in 1985, nonaquatic disposal alternatives were given only limited regard, principally because of cost considerations. Discharge of dredged material at Alcatraz is the most cost effective disposal solution for most projects. However because of its lack of disposal capacity, limiting future disposal decisions to the presently available in-Bay options is unworkable. The other two designated in-Bay sites are of inadequate size to accommodate the anticipated new work loading. As mentioned, there is presently no available ocean disposal site alternative, and the designation of an ocean site could require several more years.

The Port of Oakland recently requested certification for discharge of dredged material in the Delta at a cost far in excess of traditional disposal practices. Thus, new, more costly alternatives to the existing three disposal sites are being sought to handle dredged material disposal in the region. The DMP must consider and identify new options including aquatic, non-open water and upland alternatives that include consideration of beneficial uses. This array of disposal options must identify all feasible alternatives and will require a comprehensive management plan to insure their efficient and effective utilization.

To that end, the Dredged Material Disposal Management Program was reconfigured in early 1989. The renewed DMP was formalized in a draft "Plan Of Action" or POA, dated June 1989. The POA proposed an overall multifaceted effort to achieve a regionally acceptable LTMS. The effort was scheduled to be completed in December 1991 and cost approximately $10 million. It included studies related to: (a) management of discharged material at the existing in-Bay disposal site, including hydrographic surveying of the sites, investigations of potential suspended solids impacts and contaminant transfer, and estimates of sustained sediment yield from the sites; (b) assessment of new disposal alternatives including aquatic (in-Bay) sites, nearshore (e.g. marsh development and beach nourishment) and other non-open water disposal alternatives (e.g. permanent upland disposal and rehandling options) as well as structural/nonstructural methods for reducing maintenance dredging requirements; (c) conduct of ocean disposal site designation studies; and (d) development of administrative and analytical protocols for controlling and managing regional disposal implemented through a management plan (LTMS).

In May 1989, the South Pacific Division proposed this $10 million LTMS program to USACE Headquarters. Thereafter, a Technical Review Workshop was held in late June, chaired by Dr. Robert Engler, to critically assess the technical completeness or excesses of the program. Beyond detailed study guidance, Dr. Engler's (1989) post-workshop comments stated "... work priorities linked to fund availability is considered absolutely necessary."

In September, the Division was advised by Headquarters that the study effort should be limited to $5 million based on the Technical Workshop review and the cost of similar studies elsewhere. Furthermore, the Headquarter's guidance stated that the Corps should not be required to carry the full financial burden of the DMP. It proposed that cost sharing be pursued in accordance with the following distribution: 50 percent Corps, 20 percent Navy, 10 percent EPA and the remaining 20 percent contributed by non-Federal navigation interests. The Division was instructed to develop a revised plan in coordination with these cost-sharing partners.

## POLICY IMPLICATIONS

The legislative basis for significant cost-sharing was promulgated in the Water Resources Development Act of 1986 (Public Law 99-662), Title I. The Act broadens the financial support base for civil works projects to include affected and influenced parties. However, cost-sharing of study efforts not specifically authorized by Congress, such as the DMP, has not been formally undertaken at the national level as of yet. The initiation of formal cost-sharing agreements for the completion of the DMP lacks clear and concise policy guidance, and thus will have to undergo a local developmental process. Nevertheless, this type of participation appears to be critical to gaining the ultimate acceptability of the program's results from both the Corps' Federal as well as non-Federal constituencies.

As the State and other local navigation interests are solicited to contribute funds for the DMP, it is logical to assume their desire to have direct control over the expenditure of these funds. Part of the difficulty in revising the POA will be the incorporation of local views given the national perspective set forth in the comments of the Technical Review Workshop (Engler, 1989). For example, the Technical Review Workshop advised that long-term ambient turbidity monitoring in the Bay should be considered of less importance than new in-Bay site selection. However, the RWQCB has proposed a new resolution that would not only require monitoring of turbidity but would also limit the timing and quantities presently being discharged into the Bay. In short, turbidity is on the local Board's agenda, but new sites are not. Furthermore, Dr. Engler's comments from the Technical Review Workshop also stated, "There should not be contaminant issues of future disposal as only acceptable material will be disposed."

On the other hand, the State Water Resources Board (discussing contaminants in dredged material) has proposed "...in-Bay dredge spoil disposal has a significant and an adverse impact on the waters of the state..." and "... requests the U.S. Army Corps of Engineers to provide the State Board with an assessment of the impacts of in-Bay disposal..." (SWRCB, 1989). This State perspective of environmental concerns with respect to turbidity and contaminants differs from the national perspective as set out by the Technical Review Workshop. The Federal Standard (33 CFR 335.7) provides for such disagreements by stating that the additional cost of a plan, that would not otherwise be implemented by the Army, must be paid by a non-Federal sponsor. How this policy will be utilized in the revision of the DMP POA has not been finalized.

The problem of resolving some of these issues may be muted in the near future. The State legislature has recently passed Senate Bill No. 475 (3 August 1989) directing the SWRCB to identify toxic hot spots in California bays and estuaries and to develop sediment quality thresholds to control dredging of toxic areas. Another State agency, the Bay Conservation and Development Commission (BCDC), in charge of coastal zone management for the Bay, recently (7 December 1989) held a public hearing to review a staff recommendation for proposed legislation to prepare a State Dredging Plan for San Francisco Bay. The Commission tentatively agreed with the staff recommendation, and the proposed legislation is being drafted. In fact, the BCDC proposal is not significantly different from the DMP except that it provides for local control.

Technical disagreements may be easy to resolve compared to constraints set because of political-driven solutions. The political realities established by the furor created by fishermen over the Port of Oakland ocean disposal attempt and the Alcatraz turbidity issue is an example. In late April 1989, California Assembly Member Ted Lempert and fourteen other members of the legislature introduced a Joint Resolution, No. 43, "...to prohibit the disposal of any dredged material harmful to fishery resources or fishing activities on the continental shelf and to select disposal sites off the continental shelf or at depths greater than 1,000 fathoms." The bill has not passed through the legislature, however the intent is clear -- the legislature will set the technical agenda given the political realities of their local districts.

The implications of all these initiatives for the completion of the DMP and their impact on the San Francisco District in developing a new disposal policy for the region are impossible to foresee. However, the issues of cost-sharing, local technical perspectives and politically-driven constraints are present-day realities that must be acknowledged and dealt with in order to successfully comply with the Corps'

Congressionally authorized mission of maintaining and improving the navigable waters of the United States.

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# RISK-COST ASPECTS OF SEA LEVEL RISE AND CLIMATE CHANGE IN THE EVALUATION OF COASTAL PROTECTION PROJECTS

by

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## INTRODUCTION

The impending threat of climate change and sea level rise has brought calls from various sectors for governmental institutions to prepare for this creeping natural hazard. The U.S. Army Corps of Engineers is one such Federal agency that is responsible for various aspects of a diverse water resources and shoreline protection program. The Corps recognizes that its activities are likely to be affected by the hydrologic, meteorologic, and oceanographic consequences of global warming and expected climate changes. One response has been the explicit introduction of risk analysis to aid in the evaluation and selection of alternative plans and project components to deal with natural hazard extremes and the mitigation of their social and economic consequences. This formal risk analysis is merely an addition to an existing multi-objective evaluation procedures that guide Federal water resources development. These procedures are based on a body of social, economic, environmental, planning and decision theory literature that has been developed over the last 50 years.

There are many uncertainties and unknowns associated with the physical effects of greenhouse warming and anticipated sea level rise. These compound the existing difficulties of planning under conditions of uncertainty regarding future growth, economic development and environmental effects. The discussion will present the key risk and uncertainty issues of present-day coastal protection project analysis under uncertainty. These issues will guide the incorporation of emerging natural hazards such as climate change and sea level rise. The key issues affecting evaluation are the following:

a. Establishing the proper baseline for evaluating physical effects and socioeconomic impacts,

b. The rate and magnitude of sea level rise,

c. Uncertainty of storm frequency and wave regime under climate change, and

d. Dominant effect of the discount rate.

Weather changes associated with global warming could imply increased variability and intensity of individual coastal storm events. This phenomenon, combined with sea level rise, would further exacerbate the present conditions of beach erosion and property damages in coastal areas. Sea level rise alone, even with the present weather regime, will logically cause the landward retreat of the shoreline following the Bruun rule (Schwartz, 1967). Any change in the intensity and frequency of storm events, the direction of which is presently largely speculative, will either accentuate or diminish the physical consequences of sea level rise. An additional factor in the proper selection of strategies for societal adaptation to sea level rise and storm frequency is their rates of change. The immediacy of the consequences to shorelines and coastal development will influence the choice of action.

A fundamental question that climate change and sea level rise poses for society is how to effectively cope with the changes that appear irreversible. Many Federal, state, and local institutions are currently debating the possible strategies and specific measures for anticipating the most severe consequences

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and adapting to the inevitable changes. The Corps of Engineers, as one of these institutions, can effectively deal only with protective measures. This paper deals with how the Corps' economic evaluation principles and decision rules influence the choice of a particular shore protection measure in a risk analysis framework.

There are many other effective alternative management measures, residing within the responsibilities of the states and local communities, that should be strongly considered in adapting to sea level rise. The range of public measures to mitigate the potential hazards to life and property from sea level rise and climate change will be the same as those available today under "normal" conditions. The probable difference will be that the emphasis on alternative management strategies will change to reflect the reality that the baseline condition is changing. Thus, it is likely that shore protection strategies will shift from protective measures such as groins, bulkheads, sea walls, and beach nourishment to adaptive land use modification measures. These adaptive strategies would consist of limiting investments in and subsidies to hazard prone areas through regulation and disinvestment strategies such as transferable development rights and the use of financial incentives and tax deductions.

## AN OVERVIEW OF ECONOMIC EVALUATION PRINCIPLES

The Federal government has a long history of planning coastal protection projects. By providing protection against the hazard, efficiency gains can be achieved resulting in an increase in the national output of goods and services. There are also additional regional and local economic gains that result from the transfer of economic activity from some other location. The identification and measurement of the national efficiency gains follows benefit-cost analysis (BCA) procedures developed, to a significant extent, to evaluate the national economic implications of Federal investments in what are inherently local water resources projects. These procedures are codified in the Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies (P&G), (WRC, 1983).

Adaptive responses to sea level rise are generally the same as those considered for existing coastal erosion problems. These can be classified into four approaches or options:

**Hard engineering options** - - bulkheads, groin fields, sea walls revetments, and the elevation of the shoreline and structures,

**Soft engineering options** - - beach nourishment and dune stabilization,

**Management options** - - set back requirements, development restrictions, and land use controls, and

**Passive options** - - no systematic response, allowing coastal erosion to occur with private attempts to protect individual property.

Coastal protection projects, like all investments, involve spending money today to gain predicted benefits in the future. In addition, many types of projects, particularly the beach nourishment, maintenance type, require the commitment to future spending to maintain the project. This future aspect requires that the current and future dollar costs and benefits must be commensurate or compared in a common unit of measurement; typically in terms of their present values or the average annual equivalent of the present values. Therefore, the discount rate used to determine the present values influences the economic feasibility of alternative projects. As is well known, large discount rates reduce the influence of future benefits and costs on present values. High interest rates generally favor the selection of projects with low first costs but relatively high planned future maintenance expenditures over those with high first cost but low future maintenance expenditures.

The standard for the identification and measurement of economic benefits from a water resources investment project is individual willingness to pay (WTP). For coastal protection projects, this value can be generated by a reduction in the cost to a current land use activity or the increase in net income possible at a given site. A project generates these values by reducing the risk of storm damage to

coastal development. Conceptually, the risk from storms can be viewed as incurring a cost to development, i.e. capital investment, at hazardous locations. Thus, the cost per unit of capital, invested at risky locations, is higher than at risk-free locations. Economic theory predicts that the risk of storm damage results in less intensive development and lower land values compared to the same location with a lower risk or otherwise equivalent, risk-free locations. The risk component of the marginal cost of capital is composed of the expected value of the per unit storm damages plus a premium for risk. This risk premium results from the attitudes or preferences of the individual decision-maker toward risk. If the individual is risk averse, the risk premium is positive indicating that capital must earn a return to cover not only expected storm damages but also to compensate the capital owner for the risk.

## NATURAL SOURCES OF RISK AND UNCERTAINTY

Storms produce damage to property in coastal areas from several causes. In addition to direct wind related damage, which is ignored here, a storm typically produces a surge that raises the water surface elevation well above the mean high tide level. This wind-driven surge may be sufficient, even in the absence of waves, to produce flooding in low-lying areas from standing water. In addition to the surge, storms also produce larger waves. Property subject to direct wave attack can suffer extensive damage to the structure and contents as well as foundation erosion threatening the stability of the entire structure. Storms also produce at least temporary physical changes at the land-water boundary by eroding the natural beach and dune that serve to buffer and protect the shore and property from the effects of storms. Increased wave energy during storms erodes the beach and carries the sand offshore. At the same time, the storm surge pushes the zone of direct wave attack higher up the beach and can subject the dune and structures to direct wave action.

Several components of coastal project evaluation are stochastic so that the evaluation can be computationally complicated. For instance, the damages from storms are dependent on characteristics, described in probabilistic terms, such as intensity, duration, wind direction, and diurnal tide level. Since these characteristics, in turn, influence the level of storm surge and significant wave height, these two direct factors in storm damage are also stochastic. Sea level rise can be considered as a shift in the base elevation for measuring storm surge and wave height.

## THE EVALUATION FRAMEWORK

The establishment of a baseline or "without" project condition for incorporating physical effects and socioeconomic impacts into coastal protection measures is crucial in any evaluation. In the deterministic approach, a single forecast defines physical, developmental, cultural, environmental, and other changes. These changes are considered to occur with certainty in the absence of any systematic adaptive measure. This approach does allow, however, for individual property owners to respond to storm and erosion threats through the construction of protective measures or by property abandonment. The baseline requires assumptions to determine when these responses occur. In the risk analysis approach, this simplistic determination of the "without" condition is modified to incorporate uncertainties about storm frequencies, the distribution of wave heights, and the geomorphologic changes and property losses produced by storms and waves.

The variable <u>sea state</u> is measured as the sum of the level of storm surge plus the significant wave height. The level of storm surge is a function of the storm characteristics so that the annual probability of storm surge exceeding some level depends on the annual probability of storms that can generate a surge of that level or greater. The distribution of wave heights from a storm is not independent of the level of storm surge (Bakker and Vrijling, 1981). One can consider the storm surge to shift the probability density function for significant wave heights.

The final component for incorporating classical risk-analysis techniques within the benefit evaluation framework for storm protection, specified by the P&G, is to estimate the "with" and "without" project future economic development and land values. Without a public coastal protection project, property owners are presumed to repair structural losses with the damages from storms presumed to be capitalized into the value of the land. In addition, property owners are assumed to construct individual

protection structures when the costs are less than the value of the preserved property and the avoided expected damages to improvements.

With the project, landowners realize increases in economic rental values of land at protected locations. This rental value increase is typically considered to be equivalent to the annualized expected present value of avoided property losses with the project or the avoided costs of individual protection structures. The time stream of these benefits will reflect the stochastic nature of storm events. An important additional consideration stems from the chronological order of storms and damages. A large storm may result in damages that are so extensive that the buildings are not or cannot be rebuilt. Therefore, succeeding storms will inflict smaller losses if preceded by large storms.

The general description of the evaluation framework does not explicitly incorporate long-term shoreline erosion. In many situations, the observed shoreline retreat is simply the by-product of the storm history at a particular location perhaps in combination with relative sea level rise. In other special cases, coastal structures such as groins and jetties may induce sand starvation in down-drift areas. Typically, these are incorporated in project evaluation based on the average rate of historical shoreline retreat. For purposes here, any shoreline retreat is treated as storm-induced.

The increase in rental value of land is location based resulting from a reduction in the external costs imposed by storms and it represents a national economic development (NED) benefit as required under P&G. It is this type of economic benefit that is compared to project costs to determine the economic feasibility of any proposed Federal project.[1]

Benefits produced by a project depend on the type and scale of the protection project. Even where two alternative projects have the same scale, as defined by the design level of storm protection, (e.g., 100-year storm or probable maximum hurricane), the impact on benefits will differ depending on the magnitude of residual losses from storms that exceed the level of protection. Consider two types of projects; one a structural type, such as a sea wall, and the other a maintenance type, such as beach and dune restoration, stabilization, and periodic nourishment. For a given level of protection, the sea wall is likely to result in different residual storm losses compared to beach and dune restoration, stabilization, and nourishment.

In addition to NED benefits, a second major consideration in applying benefit-cost analysis concepts to determining the choice of project and level of protection is the stream of future project costs. The appropriate costs used in the analysis should provide a measure of all the opportunity cost incurred to produce the project outputs. These NED costs may differ from the expenses of constructing and maintaining the project. For coastal protection projects, expenses would include the first costs of project construction, any periodic maintenance costs, and future rehabilitation costs. In addition, the project may incur environmental or other non-market costs whose monetary value can be imputed. The nature of the stream of future costs depends on the type of project. For instance, a structural-type project typically has high first costs and high future rehabilitation costs but low future periodic maintenance costs. On the other hand, a maintenance-type project is composed of relatively low first costs but with larger recurring future maintenance costs.

Each of the time streams of costs must be converted into present value terms using the prevailing Federal discount rate. Note that the stream of future costs for both types of projects, but especially the maintenance, must be defined in probabilistic terms. The realized amount and timing of maintenance and rehabilitation expenditures depends on the number and severity of storms experienced at the project site in the future. Thus, the expected future cost stream is based on the estimated probability density function for sea states.

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[1] In some cases, it may be determined that there is "no Federal interest" and no Federal project. This may be the case where a "few" large identifiable beneficiaries could organize to pay for their own protection, financed out of increased land values.

Once the alternative formulated plans are evaluated in economic terms, the expected net benefits can be calculated. Following the project selection criteria in P&G, the recommended type and scale of plan should be the one which "reasonably maximizes" net NED benefits. This is a key conceptual point in risk analysis: the net benefits decision rule for selecting the economically optimal project simultaneously selects the degree of protection and level of residual risk bearing. Thus, by varying the scale of project for each type, a benefit function can be derived for each type of project. Deviations from the NED plan can be recommended to incorporate risk and uncertainty considerations in addition to the explicit risk analysis used in the economic evaluation. These could be considerations for human health and safety or non-monetized environmental concerns.

## CLIMATE CHANGE AND SEA LEVEL RISE

Thus far the evaluation and selection of Federal coastal protection investments has assumed stationarity of climate and mean sea level. The underlying physical parameters and relationships yielding the historically observed distribution of sea states have been assumed to be constant. Most forecasts for sea level rise suggest that it is not an immediate problem for coastal development. In addition, there is a wide variation in the estimates of the rate of sea level rise. For instance, a recent National Research Council report (NRC, 1987) notes that relative sea level rise is composed of two components: (1) the localized land subsidence or uplift, and (2) a world-wide rise in mean sea level. The NRC report adopted equations resulting in the following relationship to forecast total relative sea level rise:

$$T(t) = (0.0012 + M/1000) \cdot t + b \cdot t^2$$

where

$M$ = the local subsidence or uplift rate in mm/year, and

$b$ = the ecstatic component of relative sea level rise by the year 2100 in m/year$^2$.

The value of $M$ is fairly well established for many coastal locations: the value of $b$, however, is subject to wide forecast differences. Table 1 shows the estimates of the total relative sea level rise at Hampton, Virginia and Grand Isle, Louisiana for the three scenarios adopted in the NRC report. The variability in the predicted sea level rise offers a case for the application of sensitivity analysis in the evaluation of project scale. In addition, the disagreement over the ecstatic component of relative sea level rise argues for projects whose scale can be staged to account for sea level rise as it occurs.

Sea level rise can be included in the evaluation of planning alternatives as a mean increasing, variance-preserving shift in the probability density function of storm surge. This results in an increase in the site cost of capital "with" and "without" each alternative. Note that a rise in sea level will likely have different impacts on the site cost of capital for different types of planning alternatives. The incorporation of higher future sea levels in project evaluation will favor the recommendation of larger, structural type projects over maintenance-type. Two additional considerations temper this conclusion, however. First, building higher levels of protection than are economically efficient, given the current mean sea level, implies that current net benefits are sacrificed. The larger levels of protection are economically efficient only at higher mean sea levels that may or may not occur in the future. Second, since the increase in net benefits for a larger scale project occurs in the future, the discounting process necessary to determine the present values of benefits and costs will reduce the influence of these future benefits on the determination of the appropriate project scale.

One way of presenting the economic tradeoffs between design project scales that have different time streams of future net benefits is to determine the breakeven discount rate for the projects. The breakeven discount rate is the interest rate that equates the present values of two streams of future net benefits. The present value of net benefits as a function of the discount rate is shown in Figure 1 for two alternative projects A and B. Project A provides the economically efficient level of protection today ignoring sea level rise while B provides a higher level of protection in anticipation of climate change and sea level rise. Notice that the present value of future net benefits for project B exceeds the present value

Table 1. Total Relative Sea Level Risk Forecasts in Meters

| t | Hampton, VA | | | Grand Isle, LA | | |
|---|---|---|---|---|---|---|
| | I | II | III | I | II | III |
| 1985 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 1990 | 0.0 | 0.0 | 0.0 | 0.1 | 0.1 | 0.1 |
| 1995 | 0.0 | 0.0 | 0.1 | 0.1 | 0.1 | 0.1 |
| 2000 | 0.1 | 0.1 | 0.1 | 0.2 | 0.2 | 0.2 |
| 2005 | 0.1 | 0.1 | 0.1 | 0.2 | 0.2 | 0.2 |
| 2010 | 0.1 | 0.1 | 0.2 | 0.3 | 0.3 | 0.3 |
| 2015 | 0.2 | 0.2 | 0.2 | 0.3 | 0.4 | 0.4 |
| 2020 | 0.2 | 0.2 | 0.3 | 0.4 | 0.4 | 0.5 |
| 2025 | 0.2 | 0.3 | 0.3 | 0.4 | 0.5 | 0.6 |
| 2030 | 0.3 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 |
| 2035 | 0.3 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 |
| 2040 | 0.3 | 0.4 | 0.6 | 0.6 | 0.8 | 0.9 |
| 2045 | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 1.0 |
| 2050 | 0.4 | 0.6 | 0.7 | 0.8 | 0.9 | 1.1 |
| 2055 | 0.4 | 0.6 | 0.8 | 0.8 | 1.0 | 1.2 |
| 2060 | 0.5 | 0.7 | 0.9 | 0.9 | 1.1 | 1.3 |
| 2065 | 0.5 | 0.8 | 1.0 | 1.0 | 1.2 | 1.5 |
| 2070 | 0.6 | 0.8 | 1.1 | 1.1 | 1.3 | 1.6 |
| 2075 | 0.6 | 0.9 | 1.2 | 1.1 | 1.4 | 1.8 |
| 2080 | 0.7 | 1.0 | 1.4 | 1.2 | 1.6 | 1.9 |
| 2085 | 0.7 | 1.1 | 1.5 | 1.3 | 1.7 | 2.1 |
| 2090 | 0.8 | 1.2 | 1.6 | 1.4 | 1.8 | 2.2 |
| 2095 | 0.8 | 1.3 | 1.7 | 1.4 | 1.9 | 2.4 |
| 2100 | 0.9 | 1.4 | 1.9 | 1.5 | 2.0 | 2.6 |

| Scenario | Scenario Eustatic Component by 2100 | b |
|---|---|---|
| I | 0.5 | 0.000028 |
| II | 1.0 | 0.000066 |
| III | 1.5 | 0.000105 |

$T = (0.0012 + M/1000) \cdot t + b \cdot t^2$

M = rate of subsidence or uplift in mm/yr
= 3.1 for Hampton, VA
= 8.9 for Grand Isle, LA

b = the rate of change in rate of growth in eustatic sea level rise for scenarios I, II, and III.
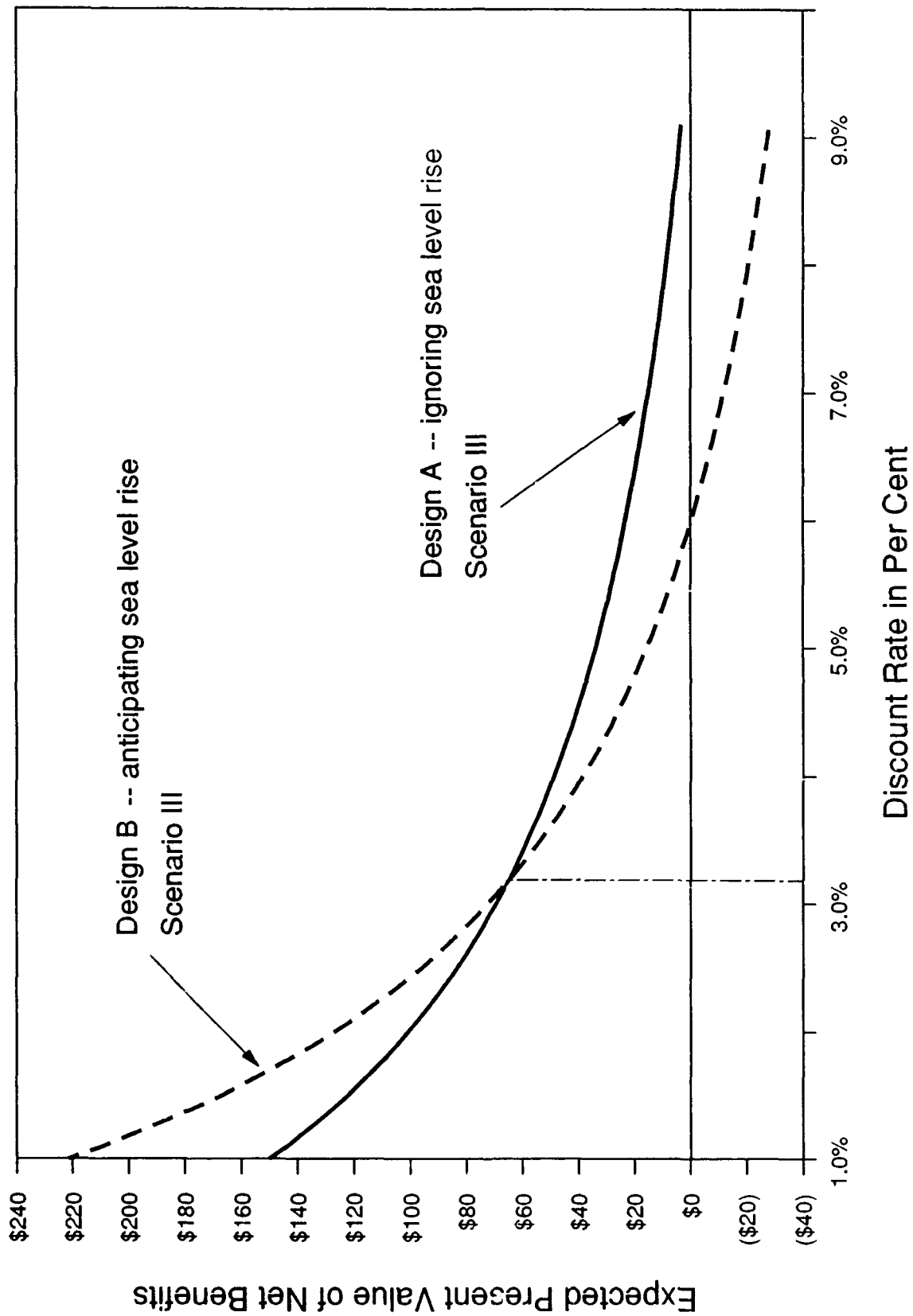
SOURCE: Based on NRC (1987).

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Figure 1.    Expected Present Value of Net Benefits as a Function of Discount Rate

of future net benefits for project A if the discount rate is less than approximately 3.2 %. This compares to the 1989 Federal discount rate of 8 7/8% used for project evaluation. In general, because sea level rise and its effects occur relatively far in the future, incorporating even a high forecast of future sea levels in the evaluation of project scale will have little impact on the economically efficient project design. Additionally, the uncertain prospect of the amount of sea level rise may support projects that are more flexible and that can easily incorporate staging of project increments as sea levels change.

Similar to the above analysis, project evaluation could incorporate the effects of forecasted climate change, expressed as a change in the frequency of storm events, through the calculation of expected values and sensitivity analysis. One hypothesis about the effect of climate change is that in many locations the frequency of severe storms will increase over time. Since recurring maintenance expenditures depend primarily on the frequency of storms, climate change that increases storm frequency will shorten the time period between these expenditures. This would tend to favor structural type projects since they have lower maintenance costs. Again, a perhaps overriding consideration for Federal projects is the impact of discounting on these future costs and their influence on project type and scale. Thus, even though climate change may result in a dramatic increase in total lifetime project costs, most of the increase occurs beyond the first 15 to 20 years of project life which has little influence on the present value of net benefits.

CONCLUSIONS

Coastal projection projects can incorporate forecasts of sea level rise and storm frequency changes due to climate change through the application of risk and uncertainty analysis techniques. The incorporation of these forecasts is not a trivial matter, but well within the probabilistic analyses currently employed to estimate project benefits and costs for coastal projects. When the effects of sea level rise and climate change occur in the future, in-place structural projects of larger scale than that warranted under the current sea level and storm frequencies would offer greater benefits than those designed for the current conditions. In addition, sea level rise and climate change which increase recurring project maintenance costs tend to favor structural-type projects.

Risk-cost analysis is not likely to yield definitive answers to the problem of choosing adaptive measures to cope with the risk of sea level rise. Other considerations which incorporate cultural, social or environmental aspects related to sea level rise may be more important in choosing adaptive measures. Risk-based approaches remind the analyst, however, that hard engineering options may exacerbate losses by encouraging development and fostering a false sense of security. Hard engineering adaptations to sea level rise, particularly the barrier type, have the potential for disaster should natural events exceed their designed level of protection. Therefore, decision makers should be wary of engineering solutions with high residual risks.

At the present time, there is considerable disagreement on the amount of sea level rise and the impact of climate change on storm frequency. More importantly, the adverse impacts on storm damages occur too far into the future, given the nature of discounting and the level of the Federal discount rate, to have much influence on the economically efficient type and scale of project recommended today. There is likely to be a greater reliance on non-structural, land use management solutions that require state and local regulatory controls. The uncertainties about the magnitude and rate of change in sea level rise emphasize the need to maintain flexibility and emphasizes the adoption of an incremental approach that preserves options.

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# NEW YORK BIGHT MONITORING, MODELING AND DATABASE STUDIES

by

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## INTRODUCTION

The New York Bight Hydroenvironmental Monitoring, Modeling and Database Studies were authorized under PL 99-662 (Water Resources Development Act of 1968), Section 728a. The goal of the five-year study was to develop a monitoring and modeling strategy for the New York Bight for use in documenting and predicting changes due to natural and human activities in the New York Bight. The study plan was developed by the U.S. Army Engineer District, New York (CENAN), Operations Division, Water Quality Compliance Branch in coordination with the U.S. Army Engineer Waterways Experiment Station (CEWES), Vicksburg, MS, and the Office of the Chief of Engineers, Dredging Division. Three CEWES laboratories are involved: the Coastal Engineering Research Center, the Environmental Laboratory, and the Hydraulics Laboratory.

In FY 89, a number of tasks were performed on initial evaluation of the New York Bight Database/Information System, Monitoring and Modeling Studies and Coordination. Much of the work centered on providing information for and holding two technical workshops on monitoring and modeling (28 and 29 June 1989, and 11 and 12 July 1989, respectively). The workshops were held to focus the studies with assistance of scientific and engineering experts in the fields of monitoring and modeling. Results of the workshops which are presented below were used in study plans for updating and refining future investigations.

The New York Bight Studies are being coordinated with the Steering Committee which is composed of Federal and state regulatory agencies: the U.S. Environmental Protection Agency (USEPA), the National Oceanic and Atmospheric Administration/National Marine Fisheries Service (NOAA/NMFS), the U.S. Fish and Wildlife Service (USF&WS), CENAN, the New York State Department of Environmental Conservation (NYSDEC), the New York State Coastal Management Program (NYSCMP), the New Jersey Department of Environmental Protection (NJDEP), and the Public Involvement Coordination Group (PICG). The CENAN is also coordinating with USEPA Region II on their New York Bight Restoration Plan, a three-year study being performed in cooperation with the States of New York and New Jersey, and on other related programs with the States and the City of New York.

The USEPA's New York Bight Restoration Plan was funded a year prior to initiation of the Corps' New York Bight Studies. Preliminary results of the New York Bight Restoration Plan were recommendations of studies aimed at restoring environmental and aesthetic uses of the New York Bight. These recommendations, formulated by coordination with scientific and public advisory groups and interagency coordination, were advantageous to the Corps because many of the immediate short-term problems in the Bight were already identified, documented, and coordinated saving both time and money in the initiation of the Corps studies.

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## NEW YORK BIGHT MONITORING & MODELING WORKSHOPS

The workshops were held at the World Trade Institute Conference Center sponsored by CENAN with contractual assistance from the Waste Management Institute, Marine Sciences Research Center, State University of New York, Stony Brook (WMI/MSRC/SUNY). Results of the workshops are presented below, together with initial specifications for model selection and improvement, requirements for model verification, the New York Bight monitoring database framework, and preliminary investigations of appropriate monitoring technologies (MSRC, 1989).

Using the New York Bight Restoration Plan recommendations as a starting point, a survey of monitoring and modeling needs (Table 1) for the New York Bight was conducted prior to the workshops. Other natural and human needs identified were longer term and/or broader in scope than could be addressed in the USEPA New York Bight Restoration Plan. For example, the USEPA effort chose to concentrate its modeling studies on hypoxia of the New Jersey coast using specific models. Their program recommended that data from the New York Bight Restoration Plan models be included in the New York Bight Monitoring and Modeling Studies. The "needs" survey, existing monitoring programs, new and developing monitoring technologies and state-of-the-art models were used as the starting point, or "strawman," for discussions held at technical workshops on monitoring and modeling. A "strawman" document was distributed to participants prior to each workshop. The workshop format included presentations followed by panel discussions and consensus building by the participants on the key issues.

## MONITORING WORKSHOP

The New York Bight Monitoring Workshop included review of historical and existing New York Bight monitoring programs and availability of data. This was supplemented by a survey of monitoring efforts performed by the WMI under contract to CENAN. Existing monitoring efforts will be used, to the extent practicable, to provide information for the New York Bight Database. In this way, duplication of effort and additional costs will be minimized. Biological, chemical, physical and geologic parameters were discussed using the "strawman" as a starting point. The status of regional monitoring programs for USEPA Region II, NOAA, CENAN, NJDEP, NYSDEC, the U.S. Army Engineer District, Philadelphia (CENAP), and the National Academy of Engineers/Marine Board Review of Monitoring Programs were presented by agency representatives. Presentations and discussions were held on monitoring techniques, database development, Geographic Information Systems (GIS), innovative technologies, satellite imagery, radiotelemetry buoys, and applicability of these methods to the monitoring needs.

Innovative technology and remote sensing were discussed in detail. The consensus was that innovative monitoring techniques should be investigated and used in the acquisition of data necessary for successful completion of modeling studies. Although remote sensing is advancing at a rapid rate, real time data from satellites will not be available at a reasonable cost for another five years. This breakthrough is expected to revolutionize data gathering for surficial oceanography. Recent advances in remote sensing using small aircraft provide a detailed, low cost alternative to more conventional surveying.

### Panel discussions

Panel discussions on biological, chemical, physical, and sediment transport monitoring were held. Consensus was reached among the technical experts and recommendations were given for the future direction of monitoring studies for the New York Bight studies. The results of the panel discussions summarized below.

### Biological oceanography

Use impairments were discussed in detail and all agreed that many complex issues need to be addressed. The actual types of monitoring and modeling will depend on the extent and geographic area of the problem. Some issues identified for further study include: the role of atmospheric deposition in

TABLE 1

## NEEDS IDENTIFICATION

NATURAL NEEDS & USE IMPAIRMENTS/MODULES (as identified by the USEPA New York Bight Restoration Plan):

    a.  Beach closures.

    b.  Unsafe seafood.

    c.  Adverse impacts on commercial/recreational navigation.

    d.  Impacts of birds, marine mammals and sea turtles.

    e.  Adverse impacts on commercial/recreational fisheries.

    f.  Loss of aquatic habitat.

OTHER NATURAL AND HUMAN NEEDS (as identified by consensus of Federal, state, and local agencies, public involvement groups, and New York Bight Workshop participants):

    a.  Dredged material disposal at the New York Dredged Material Site, inlet disposal sites, and other future disposal sites.

    b.  Woodburning at sea (being phased out).

    c.  Construction/modification of coastal structures and fill (both nearshore and offshore), i.e., *coastal flood control structures, multi-use offshore islands.*

    d.  Sewage sludge disposal at the 106-mile site (to be closed in 1991) and impacts of the former 12-mile site.

    e.  Acid waste and chemical waste disposal (being phased out).

    f.  Disposal of cellar dirt at the Cellar Dirt Disposal Site.

    g.  Coastal wastewater treatment discharges and combined sewer overflow.

    h.  Oil or chemical spills.

the New York Bight (little data exist); levels of concern for bioaccumulation should be established in accordance with applicable regulatory frameworks; methods to describe impacts on fisheries and fisheries resource species (no direct correlation exists between pollution and habitat related impacts); need for identification of key indicator organisms (re: stress, disease); identification of contaminants in seafood (species, location, real vs. perceived risks); identification of organisms to use for monitoring beach closures (fecal coliform vs. pathogens), impact of the greater metropolitan area watershed on the New York Bight, and impact of alteration of aquatic habitat (cumulative effects, eutrophication, the importance of changes in physical parameters such as suspended sediments on feeding and spawning).

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## Chemical oceanography

All agreed that it would be more productive to concentrate efforts on feasible measurements of important chemical species rather than include the entire atomic chart. Cu, Pb, Hg, and Cd levels in water and sediment were given as an example of this approach as well as organic pollutants such as CH's and PAH's, and nutrients such as N, particulates, and Si. It will be necessary to determine the boundary fluxes of metals, organics and nutrients including pulses and storm events for seasonal and spatial variability. Of key importance is knowing the quality of the data and the processes of uptake by organisms and release into other media. For example, metal transport in the atmosphere is well established, but internal recycling within organisms and the role of organics are poorly understood. This is largely due to the complexity of these relationships and the difficulty of separating effects of one chemical constituent from synergistic effects. Participants felt that the important chemical species could be identified and measured where needed and existing data could be integrated over time. In this way, a qualitative relationship between contaminants in important species in water and sediment can be established. Identification of the chemicals of concern should be based on those found in existing monitoring data. Additional species can be added, providing that "reason to believe" is established in accordance with the Federal Standard (Federal Register, 1988).

## Physical and geological oceanography

Participants all agreed physical processes operating in the Bight play an important role in the transport and distribution of materials throughout the system. Among the more important processes are: impact of wind stress on water movement, wave field characteristics, tides, influence of the Gulf Stream and the Hudson Shelf/Valley, and the interaction of Bight waters with adjacent water bodies. Particular monitoring needs of high priority include directional wave data, flux across the Rockaway-Sandy Hook transect, and suspended sediment distribution. The group felt it is essential to get a better database of mean current distributions in at least surface and bottom waters and estimates of bottom shear stresses for modeling purposes. They recommended a nested approach to modeling.

Resolution for different problems would depend upon the level of detail desired. So far, existing sediment transport models can be used for coarse-grained, non-cohesive sediments. The challenge will lie in developing or adjusting models to accommodate mixed sediment types and fluxes of fine-grained materials across the bottom. The empirical measurement of these materials will probably be a part of establishing more intricate and realistic sediment transport models.

### MONITORING WORKSHOP CONCLUSIONS

One of the most significant conclusions of the monitoring workshop was that the monitoring database will have broad and continuous application in the Bight by numerous agencies as a source for existing data for application to predictive modeling. The workshop participants agreed that collection of field data for monitoring should be performed only in response to modeling needs. Numerous monitoring programs are currently underway and large quantities of data with several to tens of years duration exist which appear to be sufficient for many aspects of model verification. Everyone agreed that a nested group of large scale hydrodynamic and water quality New York Bight models for predictive purposes are needed. Additional data should be collected only to fill in data gaps for modeling.

### MODELING WORKSHOP

The New York Bight Modeling Workshop explored state-of-the-art models of hydrodynamics, water quality, and sediment transport. An historic overview of the New York Bight, a summary of the results of the monitoring workshop and an overview of existing models in the New York Bight and adjacent areas were presented. This included a presentation on the existing Hydraulic Model of New York Harbor (located at CEWES). The New York Harbor model includes New York Harbor, the Hudson River to Hyde Park, the Raritan and Passaic Rivers and parts of Long Island Sound, Newark Bay and the Atlantic Ocean. It has been used in a hybrid model (coupled with a multi-dimensional numerical model

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of a subregion of the harbor complex) for: tidal circulation, flood control, salinity intrusion, sedimentation, pollution control navigation and coastal processes.

A presentation was given on the USEPA/Corps Chesapeake Bay Model studies. The Chesapeake Bay Models (Johnson et al., 1989; Kim et al., 1989; Dortch et al., 1989; Cerco and Cole, 1989) are state-of-the-art models which are being used to study eutrophication and related problems in Chesapeake Bay, Maryland. The model system is currently under development by the Corps and USEPA and a state/local coalition. It consists of a three-dimensional (3-D), time-varying, hydrodynamic model; a 3-D, time-varying, water quality model; and a predictive model of sediment-water interactions. The hydrodynamic model uses curvilinear coordinates which enhance model resolution of the complex Bay geometry. The hydrodynamic model is linked to the water quality model in a computational, efficient manner. The water quality model divides the Bay into a 3-D network of interconnected boxes and can run interactively with the sediment model.

The sediment model is used for description of the following processes: flux of substances between the sediments and the water column, net settling of particles to the sediment, and digenesis of organic matter within the sediment. The model system can be used to evaluate long-term changes in water and sediment quality as impacted by both point and non-point loadings.

Federal and state representatives gave presentations on their New York Bight modeling programs. Consideration was given to the use of models similar to those developed for the Chesapeake Bay Study and linkage of the existing physical model of New York Harbor to physical or computer models for the New York Bight. All of the technical experts agreed that the types of constituents of concern and transport processes in the New York Bight would be best described using state-of-the-art mathematical computer models in lieu of expanding the existing New York Harbor hybrid mathematical-physical model. However, outputs from the existing physical model were acknowledged to provide useful input across the Rockaway-Sandy Hook transect for initial testing of the hydrodynamic and water quality models.

## MODELING WORKSHOP CONCLUSIONS

The modeling workshop was divided into two panels which came to substantially the same recommendations. After a final discussion, the consensus was that hydrodynamic and water quality models similar to those being used in the Chesapeake Bay Study should be evaluated for implementation in the New York Bight. A 3-D, time-varying hydrodynamic model is needed to quantify the transport, current patterns, diffusion and dispersion and bottom velocities. Model development is needed for contaminants and sediment transport. Sediment transport models include Disposal from an Instantaneous Dump (DIFID, Johnson, 1987 and Johnson et al., 1988) and several others which are currently under development in the Corps' Dredging Research Program (DRP). The DRP sediment transport models will be used as they become available. Grid spacing in the proposed models should be closer in the New York Bight Apex area where the plethora of monitoring data has been taken over the last twenty years.

The technical experts agreed that existing modeling studies should be used to the maximum extent possible and that a nested approach should be taken. Various model classifications were discussed: far-field models such as the Chesapeake Bay model, the NOAA bedload model of the Bight, and wave models; forecasting and diagnostic models; particle tracking models (for spills); and near-field models such as the DIFID model and beach process models. The proper modeling strategy will be to link models together to develop a level of detail tailored to solve a specific problem. Model verifications and demonstrations should be performed with use of measured data where possible.

The consensus was that a regional New York Bight model(s) domain should consist of the entire New York Bight, but need not be global (i.e., the entire North Atlantic Ocean). The boundaries of the models should be the Nantucket Shoals to the north, initially the 200-meter; bathymetric contour (possibly expanded to the shelf break) to the east, the Hudson River transect to the west (between Rockaway Inlet, NY, and Sandy Hook, NY) and Cape May, NJ, to the south (Figure 1). Coupled

modeling of the Hudson River Estuary and Long Island Sound should be investigated. Models of these adjacent water bodies are being developed under other programs and may provide sufficient data to force the New York Bight regional model(s). However, it will be important to accurately represent the dynamic interaction of these areas. The Database/GIS will be used in support of the monitoring and modeling tasks. As described above, monitoring will be performed in support of the model tests on an as-needed basis. Innovative monitoring techniques will be pursued to take advantage of emerging technologies which have the potential for supplying quality synoptic data at low cost. A recommendation was made that a working group of technical experts be established for New York Bight monitoring and modeling studies similar to the advisory group for the Chesapeake Bay Study.

## SUMMARY

The results of the New York Bight Monitoring and Modeling Workshops as well as subsequent meetings with technical experts have allowed the Corps to apply scientific and engineering expertise to the needs for monitoring and modeling in the New York Bight and to focus the direction of future studies. A Database/GIS system established for the New York Bight will be valuable both for the monitoring and modeling studies and for other future studies (as recommended through Federal, state, university and public coordination). Existing monitoring data are currently being compiled, evaluated and input to the Database/GIS.

The New York Bight Monitoring Studies will also investigate innovative technologies for monitoring, including state-of-the-art radiotelemetry buoy systems and remote sensing. The New York Bight Modeling Studies will examine the applicability of various mathematical models for hydrodynamics, water quality, and transport in the New York Bight. Verification and linkage of appropriate test models will be performed to determine the models to recommend for implementation. A technical workshop is planned for late FY 90 to discuss progress to date and refine the scope of work for the additional years of study. Coordination will continue with Federal, state, and local programs to minimize duplication of effort and optimize use of the available funding for the New York Bight Studies.
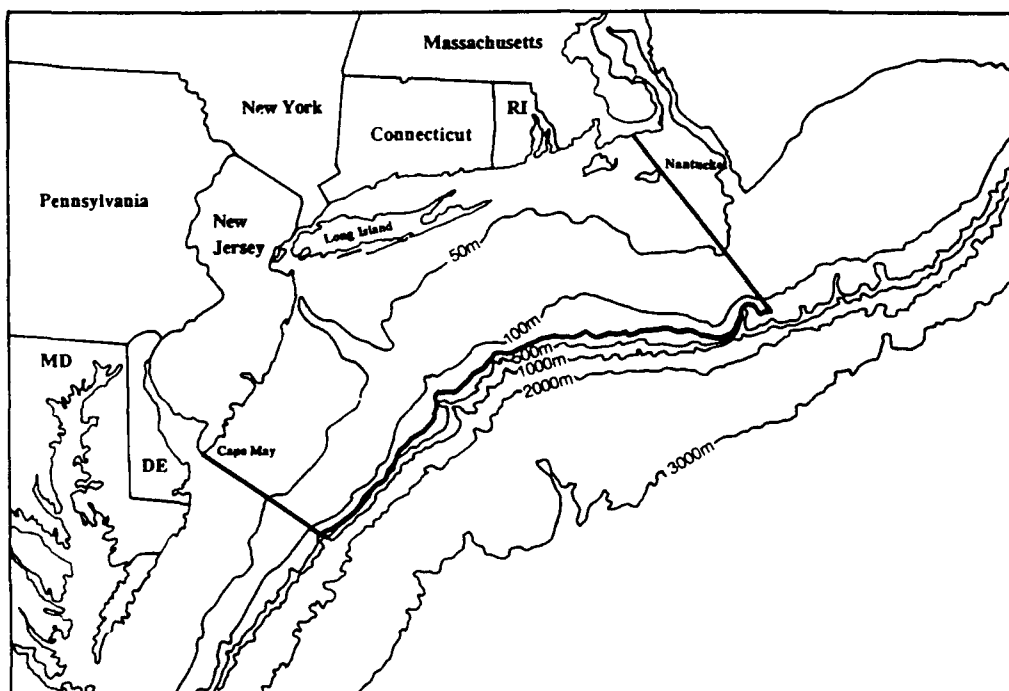


Figure 1. Proposed boundaries for New York Bight models

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# A PILOT STUDY DREDGING AND DISPOSAL METHODS
## NEW BEDFORD HARBOR, MASSACHUSETTS

by

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## INTRODUCTION

New Bedford, Massachusetts is a port city located at the head of Buzzards Bay, approximately 55 miles south of Boston (Figure 1). The harbor is underlain by sediments containing elevated levels of PCBs and heavy metals including copper, chromium, zinc, and lead. PCB concentrations in the sediment range from a few parts per millon (ppm) to over 100,000 ppm and levels in harbor water have been measured in the parts per billion range (Weaver 1982).

The source of the contamination was two electrical capacitor manufacturers who were major users of PCBs from the time their operations commenced in the 1930's until 1977. These industries discharged wastewaters containing PCBs directly into the harbor and indirectly via the municipal sewer system (EPA 1976).

In 1984 the Environmental Protection Agency (EPA) asked the Corps of Engineers (USACE) to evaluate the engineering feasibility of several dredging and disposal alternatives that had been proposed for the northern portion of the site, referred to as the Acushnet River Estuary. This 200-acre area extends from the Wood Street Bridge south to the Coggeshall Street Bridge (Figure 2) and contains the highest levels of contamination.

An Engineering Feasibility Study (EFS) was conducted at the Waterways Experiment Station which consisted of field data collection, literature reviews, laboratory studies and modeling efforts leading to the development of conceptual alternatives for dredging and dredged material disposal (Francingue 1988). The pilot study, which is the subject of this paper, is an extension of the EFS (NED 1989).

## PILOT STUDY DESCRIPTION

The study involved the evaluation of three types of hydraulic dredges and two disposal methods. Approximately 10,000 cubic yards of sediment were removed, 2,900 cubic yards of which was contaminated. Both a confined disposal facility (CDF) and contained aquatic disposal (CAD) were utilized. Operations were extensively monitored to detect chanres in water quality throughout the harbor that could be attributed to the dredging and disposal operations while gathering data to address the technical objectives of the study. These objectives included:

* determining if dredges could effectively remove the contaminated sediment,
* evaluating sediment resuspension and contaminant release at the point of dredging and during CAD,
* determining if contaminated sediment could be contained in a CAD cell.

## PILOT STUDY SITE

Dredging and disposal operations were conducted in and adjacent to a small cove located in the upper estuary (Figure 3). PCB levels in the cove were 150 - 600 ppm in the top six inches of sediment and were not detectable below two feet. Water depths were 0.5 feet at low water with a tide range of approximately 4 feet. The sediment was an organic sandy silt. The shallow water and fine grained material are typical of a large portion of the upper estuary.

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[1]Civil Engineer, Operations Division, U.S. Army Corps of Engineers, New England Division.
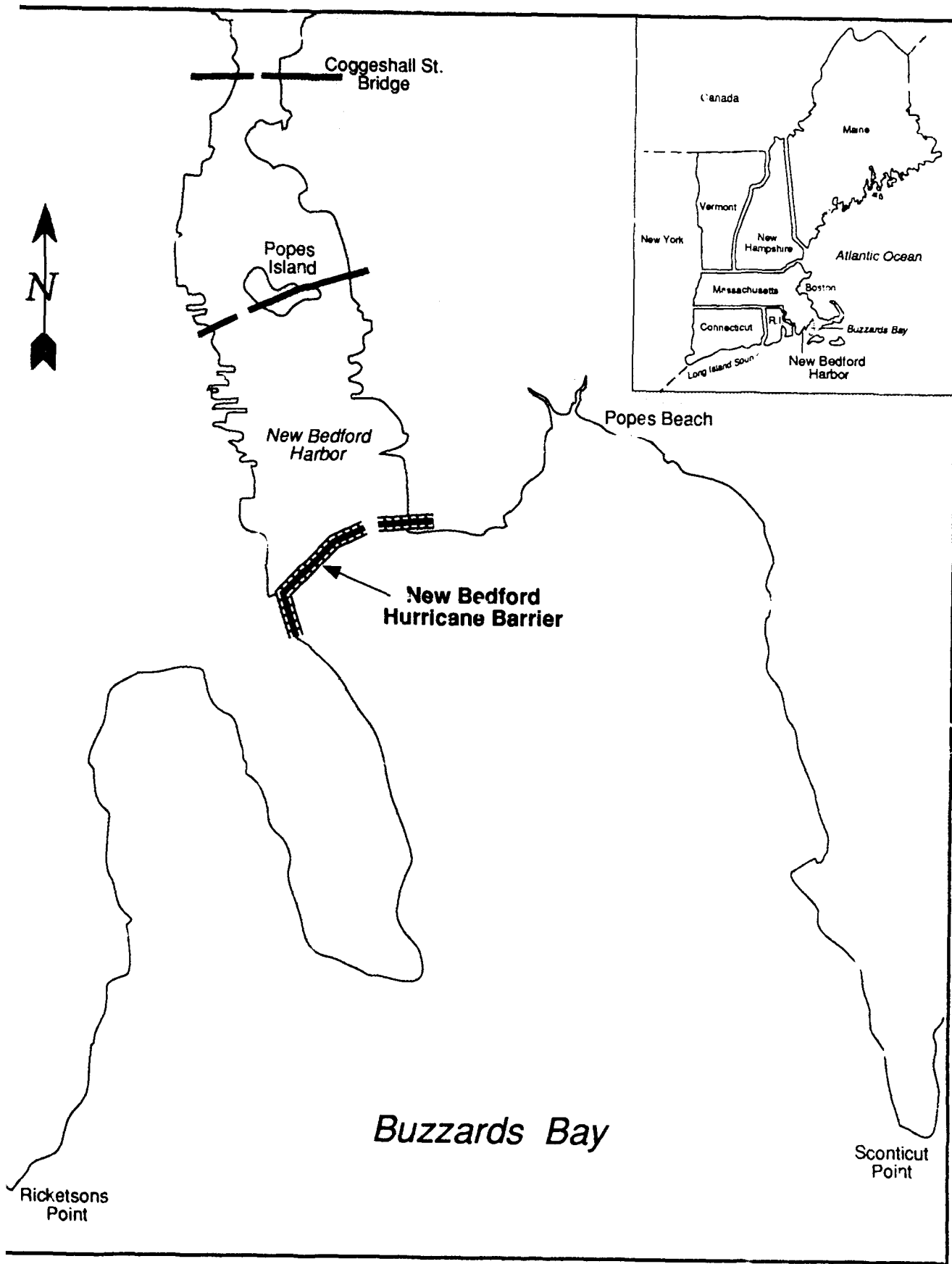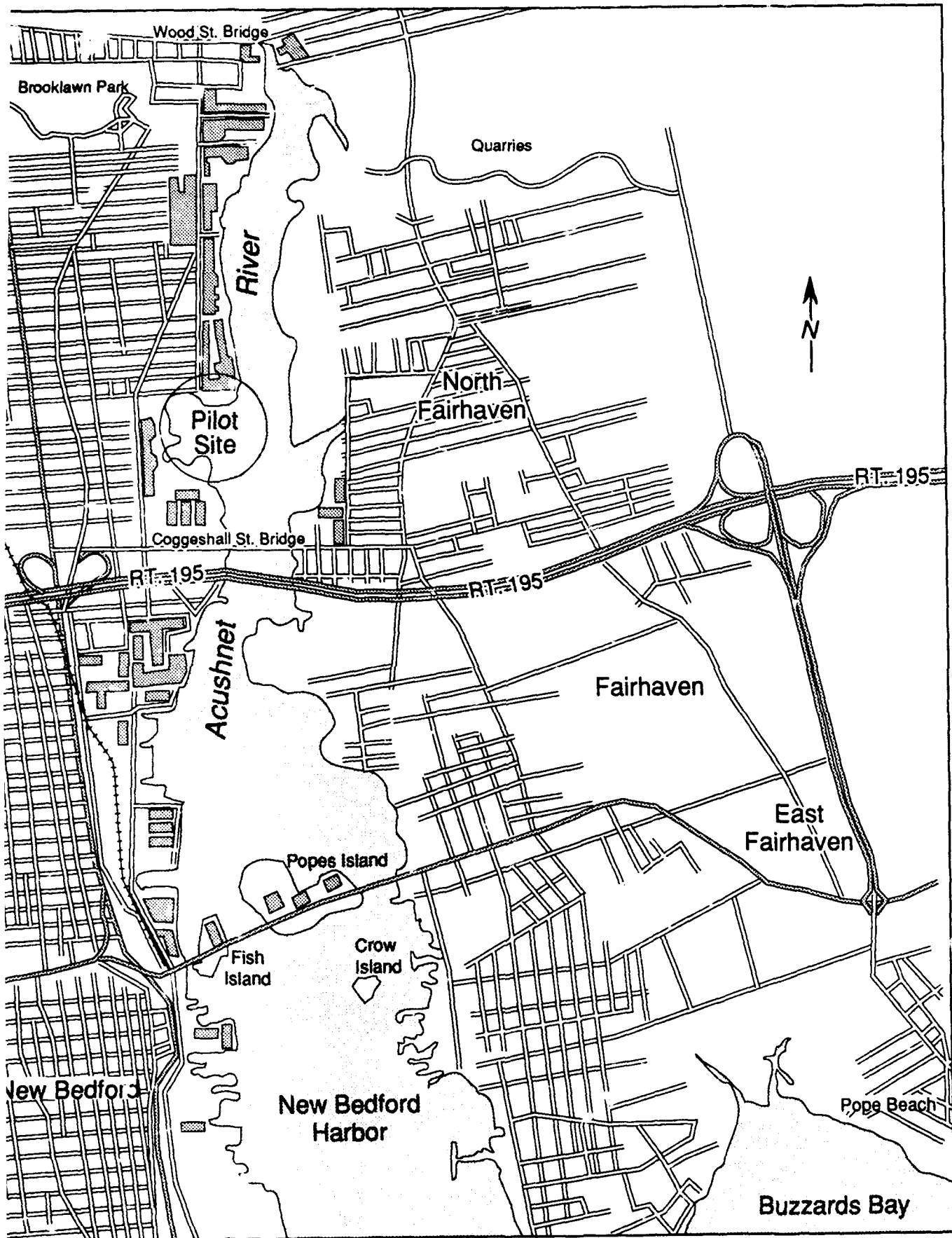
Figure 1    Outer New Bedford Harbor

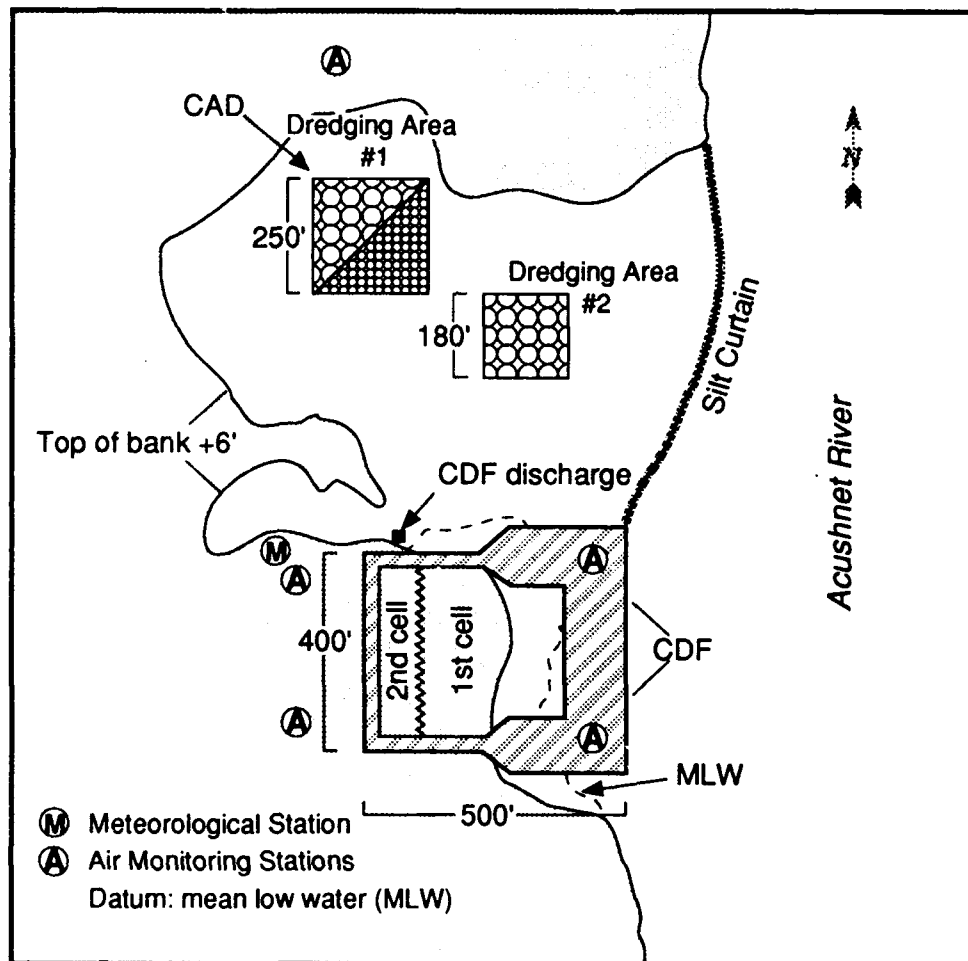Figure 2  New Bedford Harbor & Acushnet River Estuary

Figure 3  Pilot Study Site

## DREDGING EQUIPMENT

Three hydraulic pipeline dredges were used during the study; a cutterhead dredge, a horizontal auger dredge known as a Mudcat, and a dredge with a specially designed dredgehead called a Matchbox. They were selected after a thorough evaluation by USACE personnel that considered a wide range of dredging equipment. The following factors were considered critical in selecting the dredges:

* ability to remove the layer of contaminated sediment while minimizing overdredging,
* ability to minimize resuspension of sediment while operating
* ability to operate in the shallow water which is prevalent in the upper estuary.

The major operational differences between the dredges are their dredgeheads and their method of moving through an area while dredging. Both the cutterhead and horizontal auger dredges have a revolving dredgehead which breaks up and agitates the bottom sediments. The Matchbox dredgehead has no mechanical action and relies on suction forces to move the sediments. Both the cutterhead and Matchbox dredges move through an area in the same manner. The dredges pivot on their stern spuds and dredge in a zig-zag fashion while pulling on swing anchors. The Mudcat operates off a cable system, winching itself along in both forward and reverse.

During the course of the study different methods of operation were experimented with while attempting to meet the project goals of removing the contaminated sediments while minimizing both overdredging and the resuspension of sediment. Operating parameters that were adjusted included swing speed, rate of advance, rotation of the cutterhead and depth of cut.

## DISPOSAL METHODS

The two disposal methods evaluated during the study were a confined disposal facility (CDF) and contained aquatic disposal (CAD). The CDF is a retention basin that was constructed on approximately 250,000 square feet of area, half of which was located below the high water line. A granular fill dike was constructed around the perimeter of the CDF and it was divided into two chambers by a sheetpile wall. The dredged material slurry entered the primary cell where the majority of the solids settled out. The excess water flowed over a weir built into the sheetpile wall where a cationic polymer emulsion was sprayed into the flow to promote flocculation in the secondary cell prior to discharging the water back into the estuary. Approximately 2200 cubic yards of contaminated sediment were pumped into the CDF and subsequently capped with approximately 3900 cubic yards of clean dredged material.

CAD involved initially dredging a pit or cell in the bottom of the estuary. Approximately 700 cubic yards of contaminated sediment were then discharged through a diffuser along the bottom of the cell and subsequently capped with approximately 2600 cubic yards of clean dredged material placed in the same manner. The diffuser is attached to the discharge end of the pipeline and suspended just above the bottom surface. It reduces the exit velocity and turbulence of the slurry, thereby enhancing the settling of the dredged material solids. The area dredged during the filling of the CDF was used as the CAD cell. It was approximately 180 feet by 140 feet in size with the bottom elevation of the excavated cell generally less than -6 feet Mean Low Water. A 0.5 to 1.0 foot layer of contaminated sediment and a 1 to 3 foot layer of cap material were placed in the cell.

## MONITORING

EPA's Environmental Research Laboratory in Narragansett, Rhode Island assisted the USACE in designing and executing the monitoring program which consisted of physical, chemical, and biological evaluations of sediment, harbor water, leachate and effluent from the CDF. The following were the programs major objectives:

* monitor contaminant release pathways associated with the dredging and disposal operations,

* monitor water quality throughout the harbor,
* provide data in a timely fashion to assist in managing ongoing dredging and disposal operations.

Four monitoring stations were established throughout the harbor. Station 1 was located approximately 1000 feet north of the cove, station 7 was located at the entrance to the cove, station 2 was located at the Coggeshall Street Bridge and station 4 was located just north of the hurricane barrier. They were initially sampled to determine the existing ranges of specified physical, chemical and biological response variables which occur within the system. These same stations were subsequently sampled during the three phases of the project; CDF dike construction, dredging with disposal into the CDF, and dredging with CAD.

The Coggeshall Street Bridge station was the focus due to the location (boundary between the more highly contamianted upper estuary and the less contaminated lower harbor) and the fact that water circulation is restricted at this point. At this station the flow was measured for each sampling event and samples from six cross sectional sub areas were composited proportional to velocity. Equal portions of the five hourly composites were then combined to form one sample representing either the flood or ebb tide condition.

Samples from the other stations were taken hourly at 3 depths. The five hourly composites were then combined into one sample for each station which represented the ebb or flood tide condition. The results of the pre-operational monitoring are summarized in Table 1.

Contaminant release pathways that were monitored during the operational phases of the study included:

* CDF: leachate, effluent, airborne;
* CAD: sediment resuspension and contamiant release;
* Dredging: sediment resuspension and contaminant release at the dredgehead, plume development and movement, effectiveness of contaminant removal.

---

## TABLE 1

### BACKGROUND CONDITIONS

| | |
|---|---|
| seawater temperature | 18.5°C - 23.5°C |
| salinity | 24 ppt - 33 ppt |
| total suspended solids | 6.4 mg/l- 10.2 mg/l (station 2) |
| | 4.4 mg/l- 7.9 mg/l (station 4) |

### AVERAGE CONTAMINANT LEVELS

| Station | Tide | TotalPCB (ug/l) | Cd | Cu (ug/l) | Pb |
|---------|------|-----------------|------|-----------|------|
| 2 | ebb | 0.61 | 0.20 | 3.4 | 6.5 |
| 4 | ebb | 0.11 | 0.11 | 2.3 | 2.9 |

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RESULTS

The dredges used during the pilot study were able to effectively remove the contaminated sediment while minimizing the amount of material removed. PCB levels after two passes of the cutterhead and Matchbox dredges were less than 10 ppm while only a 1.1 to 1.5 foot layer of material was removed. Resuspension rates and contaminant release at the point of dredging varied but impacts 500 feet from the the point of dredging were minimal for all dredges. Average values for suspended sediment and PCBs adjacent to the dredgehead are summarized in the following table. The cutterhead dredge was recommended for future work in New Bedford based on the results of the pilot study.

| Dredge | Total Suspended Solids (mg/l) | Total PCB (ug/l) |
|--------|-------------------------------|------------------|
| Cutterhead | 82 | 7.0 |
| Matchbox | 274 | 2.6 |
| Horizontal auger | 1610 | 54.9 |

(1) Samples used to obtain this data were taken from a sampling device installed at the dredgehead. Samples were drawn from 6 sampling ports which represented 3 depths and both sides of the dredgehead.

Figure 4 shows the dredging area and the array of 15 stations which were sampled hourly while the dredge was operating. This monitoring effort was carried out to detect a plume of resuspended material or contaminants that might be moving away from the dredging area. A well-defined plume never developed, however, as conditions essentially returned to background within 500 feet of the dredge.

A summary of suspended solids sampling for a typical day of cutterhead dredge operation is shown on Figure 5. The values shown represent the average of five individual stations which make up each row. Composite samples were formed from the five individual samples (example: stations 6-10, sampling event 3) and analyzed for PCBs. The results showed a considerable decrease from the levels detected at the dredgehead and were similar for all dredges. The data from the cutterhead dredge are summarized below.

CUTTERHEAD DREDGE

Plume Sampling Stations - Total PCB (ug/l)

| Station | Mean | Minimum | Maximum | Number of Samples |
|---------|------|---------|---------|-------------------|
| 6-10 | 1.7 | 1.4 | 1.9 | 3 |
| 11-15 | 1.0 | 0.5 | 1.4 | 2 |
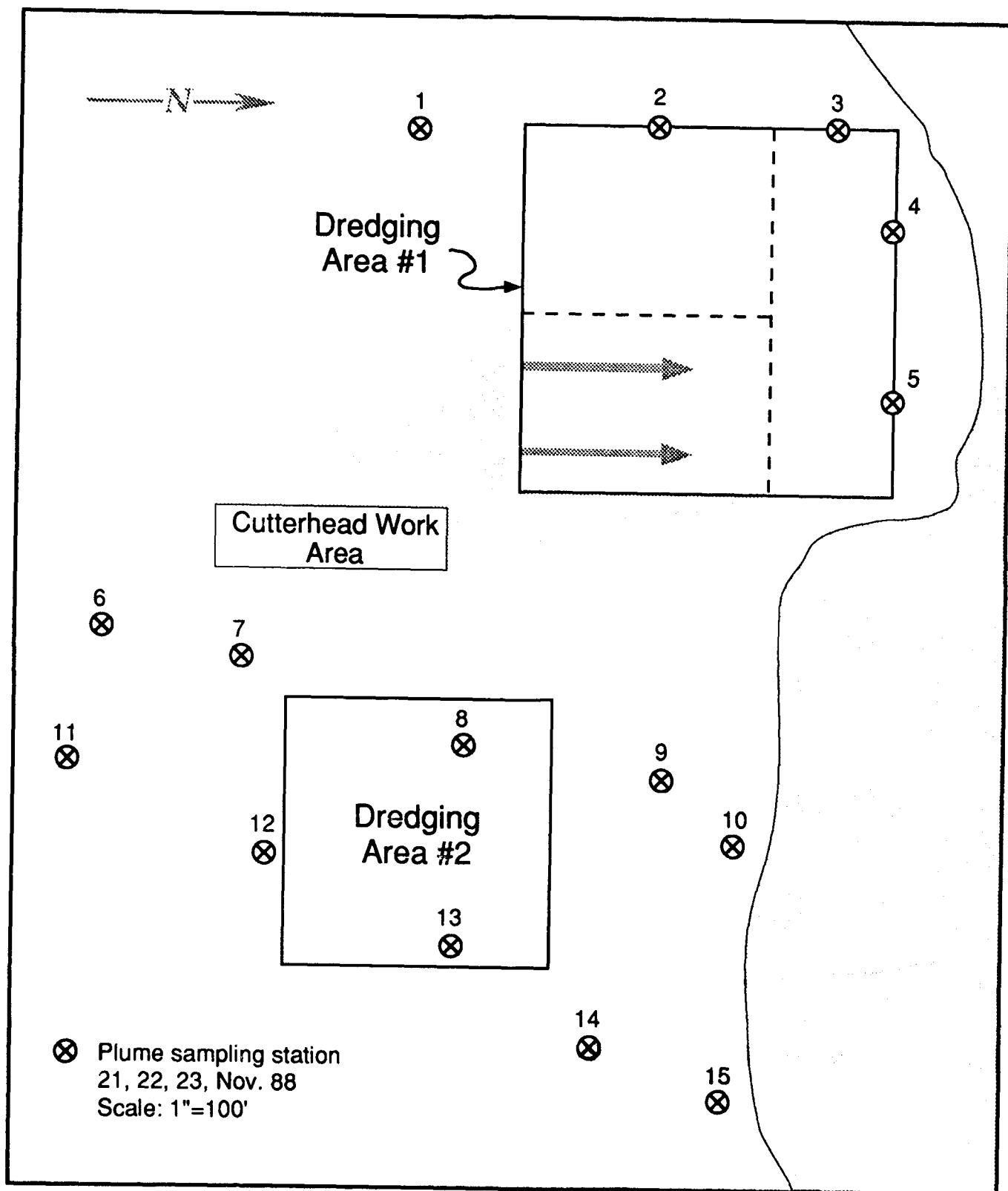
Figure 4    Plume Sampling Stations
Cutterhead Dredge

Figure 5    Total Suspended Solids - Dredging



1. The x-axis represents the time period commencing with the start of dredging
2. The values shown represent the average of 5 individual samples
3. Total suspended solids levels adjacent to the operating dredgehead averaged 146 mg/l during this period

22 November
▲ 300-400 feet
● 450-550 feet

Time (hours)

Total Supended Solids (mg/l)
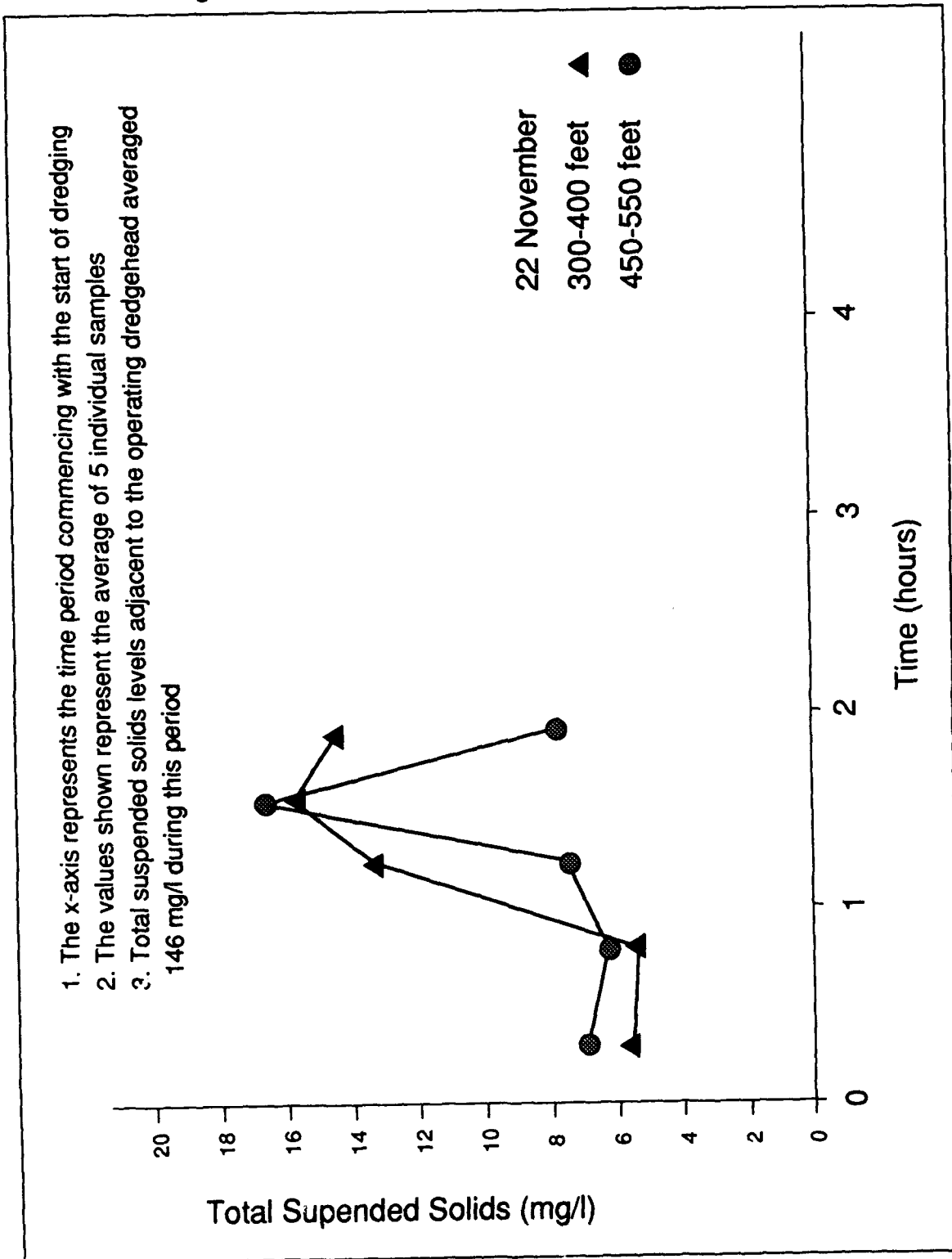
Contained Aquatic Disposal: The major questions relating to CAD addressed in this paper involve the placement of contaminated sediment within the cell and the sediment resuspension and contaminant release associated with the operation. Hydrographic survey results indicate that a one-foot layer of contaminated sediment was placed in the cell. An array of 10 stations (Figure 6) was established around the CAD cell to detect any resuspended material or contaminants moving away from the operation. The results of this effort indicated that both suspended sediment and contaminant levels were elevated above background and other phases of the pilot study. A summary of suspended solids sampling for a typical day of CAD is shown on Figure 7. Composite samples were formed from the five individual samples (example: station 6-10, sampling event 3) and analyzed for PCBs. The results are shown below.

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CONTAINED AQUATIC DISPOSAL

Plume Sampling Stations - Total PCB (ug/l)

| Stations | Mean | Minimum | Maximum | Number of Samples |
|----------|------|---------|---------|-------------------|
| 1-5 | 13.4 | 2.5 | 31.8 | 5 |
| 6-10 | 6.8 | 1.5 | 15.3 | 4 |

---

Confined Disposal Facility: Both the weir between the primary and secondary cells and the discharge from the facility were monitored during the operation. Samples were taken hourly and analyzed for total suspended solids. A daily composite was formed from these hourly samples and analyzed for PCBs and metals. The results of these efforts are summarized in the following table.

---

CONFINED DISPOSAL FACILITY

Total Suspended Solids Data (mg/l)

|  | Daily Average | Range | Number of Days Sampled |
|--|---------------|-------|------------------------|
| weir | 97 | 35 - 344 | 19 |
| discharge | 75 | 27 - 152 | 15 |

PCB Data

Dissolved PCB (ug/l)

|  | Mean | Range | Number of Samples |
|--|------|-------|-------------------|
| weir | 1.9 | 0.6-4.3 | 12 |
| discharge | 1.4 | 0.3-2.9 | 11 |

Particulate PCB (ug/l)

|  | Mean | Range | Number of Samples |
|--|------|-------|-------------------|
| weir | 9.1 | 5.2-15.9 | 12 |
| discharge | 10.9 | 5.0-19.2 | 11 |

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Figure 6    Plume Sample and Diffuser Locations
Contained Aquatic Disposal

Figure 7   Total Suspended Solids - CAD

1. The x-axis represents the time period commencing with the start of dredging
2. The values shown represent the average of 5 individual samples

8 January
100 feet
250 feet

Time (hours)

Total Supended Solids (mg/l)

This information indicates that the method used to predict the contaminant load in the CDF discharge (Palermo,1985) produces a conservative estimate at this site. This method uses laboratory settling column data to estimate the effluent suspended solids which is then combined with the results of the modified elutriate test.

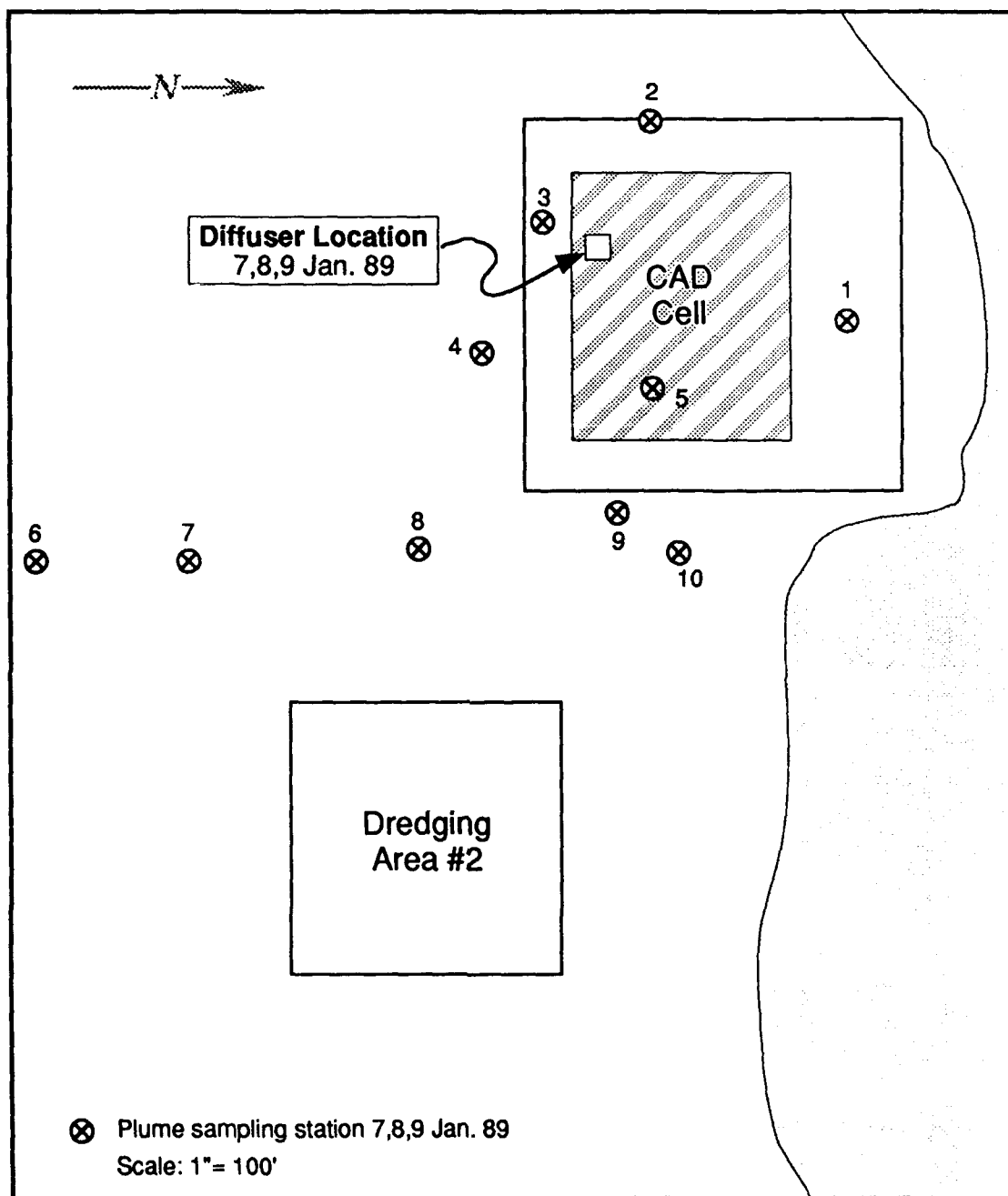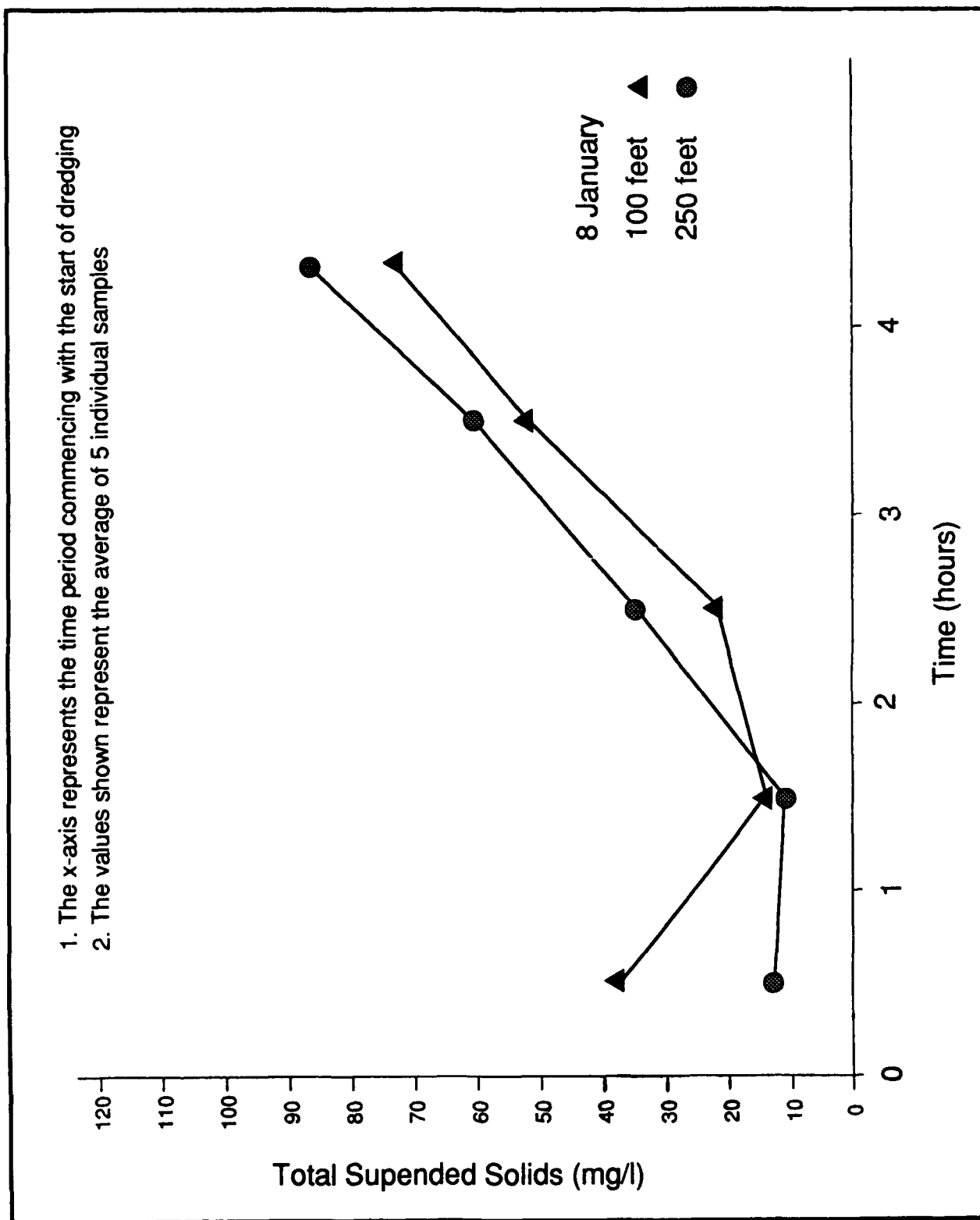Harbor Monitoring: PCB levels detected at the monitoring stations within the harbor during the operational phases of the pilot study are compared with the background conditions in Table 2. The levels detected at the Coggeshall Street Bridge (station 2) did not represent a statistically significant increase above background conditions. These results indicate that the dredging and disposal operations did not significantly increase the movement of contaminants into the lower harbor.

## CONCLUSION

The study obtained site specific data on the operational characteristics of the dredges and contaminant release and sediment resuspension associated with their operation. This allowed for field verification of laboratory techniques for predicting contaminant release and will improve our ability to make future estimates for work in New Bedford.

The study also addressed two major concerns regarding the evalaution of remedial actions for the upper estuary. It showed that dredges could successfully remove the contaminated sediments from the harbor. PCB levels in sediment remaining after two passes of the dredge were less than 10 ppm. It also showed that dredging and disposal operations could be carried out without a significant increase in the release of contaminants from the upper estuary to the lower harbor. This is evident from the results of monitoring at the Coggeshall Street Bridge (station 2) during the various stages of the study (Table 2).

The study, along with previous work performed during the EFS, has allowed the Corps of Engineers to make specific recommendations to EPA concerning future dredging work in New Bedford. EPA will rely on these recommendations in arriving at their decisions on remedial actions at this site.

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## TABLE 2
## HARBOR MONITORING STATIONS

### Total PCB (ug/l)

| Phase | Tide | STATION 1 | | STATION 2 | | STATION 4 | | STATION 7 | |
|---|---|---|---|---|---|---|---|---|---|
| | | Average | Range | Average | Range | Average | Range | Average | Range |
| 1 | ebb | 1.7 | 0.4–3.8 | 0.6 | 0.4–0.8 | 0.12 | 0.05–0.22 | | |
| 1 | flood | 1.2 | 0.8–2.0 | 0.5 | 0.3–0.7 | 0.10 | 0.05–0.13 | | |
| 2 | ebb | 1.5 | 0.6–3.2 | 0.9 | 0.4–1.5 | 0.11 | 0.05–0.22 | | |
| 2 | flood | 1.1 | 0.6–2.4 | 0.4 | 0.2–0.7 | 0.08 | 0.07–0.09 | | |
| 3 | ebb | 1.4 | 0.8–2.0 | 0.7 | 0.4–0.9 | 0.11 | 0.10–0.12 | 0.9 | 0.87–0.94 |
| 4 | ebb | 1.1 | 0.4–1.2 | 0.5 | 0.3–1.0 | 0.12 | 0.06–0.23 | 0.8 | 0.3–1.9 |
| 4 | flood | | | 0.2 | 0.2–0.3 | 0.10 | 0.02–0.21 | | |
| 5 | ebb | 1.5 | 1.0–2.4 | 0.9 | 0.6–1.5 | 0.15 | 0.09–0.22 | 2.6 | 0.9–5.4 |
| 5 | flood | | | 0.8 | 0.6–0.8 | 0.16 | 0.12–0.19 | | |

Phases:
1. Preoperational
2. Dike Construction
3. Pre Dredging
4. Dredging
5. CAD

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# ESTUARINE BOUNDARY LAYER INSTRUMENT SYSTEM

by

Thad C. Pratt[1]

## INTRODUCTION

Estuarine Boundary Layer Instrument System (EBIS), has been designed at WES under the U.S. Army Corps of Engineers' General Investigations Research Program. The intended purpose of EBIS is to measure near-bed phenomena in the estuarine zone. Parameters to be measured by the instrument array include velocity, shear stress, salinity, temperature, conductivity, and turbidity. Through the course of research it was determined that one array design could not be used in all applications. With this in mind an array for deployment in low-energy sites was started.

## EBIS SYSTEM DESIGN

Technology for near-bed measurements in tidal waters has been in development for many years. The work presented here is intended to adopt and advance that technology for use in the Corps of Engineers applications.

### STRUCTURE

Structural design considerations of the system include, ease of deployment, rigidity, stability and system durability. The initial structural design was very similar to the 1978 Sediment Water Interface Probe (Figure 1) (Nichols, et al. 1978). Modifications were made to the EBIS system to allow for disassembly and reassembly of the array. This modification greatly improved ease of transportation to the job site and deployment.

Deployment site characteristics dictate size limitations and profiling needs in a survey. With this in mind, different length legs (2, 3 and 5 feet) were constructed to allow for variable profiling depths. Various bottom types for example, sand, mud and gravel, dictate foot design criteria. For the hard sand and gravel bottoms small hinged feet were constructed, but in the case of a muddy bottom these feet are not adequate to support the structure. Wider and larger radius feet were constructed to distribute the structural weight over a larger surface area, reducing the amount of settling into the muddy bottom and increasing structural rigidness. These larger feet are to be used in high flow deployments to increase array rigidness regardless of bottom types.

### INSTRUMENT POSITIONING

A DC-powered rotary actuator driving a linear actuator with a 5-foot stroke gives the array profiling capabilities. The rotary actuator is housed in an aluminum pressure housing in the approximate center of the tripod. The various probes are connected to a T-bar which is oriented perpendicular to the ram (Figure 2). Presently, the rotary actuator is controlled and powered from the surface, and the position of the T-bar is recorded by the data logger from a position potentiometer inside the actuator housing. The main reason for surface control is due to electrical noise produced by the rotary actuator which would interfere with the data logger. The actuator also requires a large 12-VDC power supply in addition to the surface supplied power.

A limitation with this design is that the system height is almost 12 feet with the 5-foot legs, thus limiting deployment of the array to larger vessels. A smaller hydraulic operating system is being

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[1]Physicist, Hydraulics Laboratory, Estuaries Division, U.S. Army Engineer Waterways Experiment Station.

Lifting Cable

Data Logging Cables

SEDIMENT-WATER
INTERFACE PROBE

1-Nuclear transmission density probe
2-Optical turbidity meter
    Ranges a. 0-100 ppm
            b. 0-20,000 ppm
3-Electromagnetic current meter
4-Pressure transducer
5-Hydraulic cylinder
    (a) stop bolt
6-Support brackets
7-Pads
8-Lead weights
9-Directional fin
10-Pump intake

Figure 1  Sediment Water Interface Probe
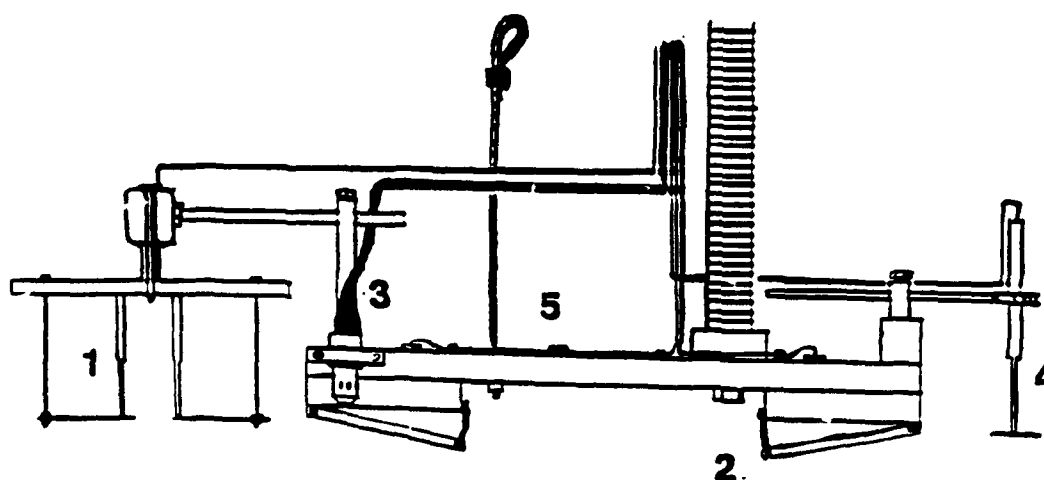
1. Bed Shear Stress
2. Bottom Detector
3. OBS Sensor
4. Flow Sensor
5. Temperature

Figure 2 T-Bar

developed to give the same profiling capabilities, and reduce the tripod height to approximately 6 feet, which will allow for deployment from a smaller vessel.

DATA ACQUISITION AND CONTROL

In the over all design of the EBIS system, long range deployment is a primary goal. In order to take long-term data sets of dredge material disposal sites and channel shoaling sites. To allow this type of monitoring, the data logger and power supply had to be mounted on the array. Waterproof cylinders were designed to house the power supply and data logger. These pressure housings are mounted approximately in the center of the array. In the initial prototype design these housings were made larger than necessary to allow for future system modifications, but upon system completion the housings and power supply will be reduced to a minimum size in order to reduce flow drag on the structure. During initial design and development, connection to the surface is maintained for monitoring purposes.

The data loggers used are Tattletale Models 5 and 6. The Model 5 has built-in BASIC on the mother board to allow for easy programming and fast execution. It has unlimited 10-bit A/D channels, and unlimited digital I/O channels. The Model 5 has 2 megabytes of data stor. ge capabilities in RAM. The Model 6 is configured the same as the Model 5 except it has 12-bit resolution per channel and a 20-megabyte hard drive for data storage. Both data loggers have low power consumption, less than 100 mA, which helps to reduce the total power requirements.

SENSORS

Probes mounted to the array in the initial development stages include a vector measuring velocity meter (VMVM), pressure gage, flux gate compass, two inclinometers, temperature probe, and an optical backscatter turbidity sensor. After further testing and array modifications, the VMVM was replaced with two specially designed, constant temperature, metal-clad flow sensors and a separate array of 5 electromagnetic velocity meters. A bed shear stress sensor, a second OBS turbidity sensor and a more accurate temperature transducer were also added to the sensing capabilities. The following paragraphs provide more detailed explanations of the sensors used in the instrument array.

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## VECTOR MEASURING VELOCITY METERS

The VMVM was developed at WES in an attempt to provide capabilities unavailable commercially. They are designed to work in high-suspension environments where electromagnetic velocity meters fail. The WES VMVM propellers are small in diameter allowing measurements close to the bottom. They were designed to respond only to velocities parallel to the axis of the rotation. This feature is commonly known as cosine response. The axis of rotation is horizontal to the bed, and the minimum threshold limit of the meters is 0.1 ft/sec.

## PRESSURE

The pressure gage mounted in the data logger pressure housing is a stainless steel strain gage type cell with a range of 0-50 psia. This cell allows the array to operate in a maximum of approximately 60 feet of water. This sensor gives an accurate record of tide-induced water surface fluctuations during the deployment period.

## COMPASS

A flux gate compass is mounted in the instrument pressure housing to give the relative orientation to magnetic north. This measurement is necessary in calculating the direction of flow from the velocity sensor outputs, and to give a relative orientation of the array.

## INCLINOMETERS

Inclinometers are mounted in the instrument housing measuring the tilt of the array as it sits on the bottom. The array orientation will prove useful in examining data sets that have unusual values. It is also a useful measurement in determining the deployment site validity. The sensing capabilities of the inclinometers is + 45 degrees from the mounting plane. The sensors are mounted 90 degrees apart on the x and y axis of the tripod structure.

## TURBIDITY

Suspended sediment is measured by using the Optical Backscatter Sensor, OBS sensor. This sensor measures the amount of infrared light reflected back from the suspended sediment particles. This sensor has a very linear response, but it has to be calibrated for every use with sediment types that are characteristic to the deployment site. The reason for this is that the calibration is grain size dependent, and the material could change from location to location. The maximum sensitivity range for mud is 5000 mg/l and 100,000 mg/l for sand. The most important feature with these sensors is that they can be adjusted to give maximum resolution for a defined range of concentrations.

## BOTTOM DETECTION

A bottom detector is mounted on the T-bar. The detector consists of a hinged aluminum plate with a known surface area and density. When the bottom material exerts enough upward force on the plate to overcome its submerged weight, a switch is tripped and the position is recorded. One detector is mounted on each end of the T-bar in case the bottom is not level. These detectors are designed so that different types of plates can be used to detect the bottom, allowing different density bottoms to be detected. In the future, a cone penetrometer might be incorporated into the system to measure the penetrating force of the bottom.

## BED SHEAR STRESS

A bed shear stress probe was developed under a contract with the University of South Florida (Gust,1988). It uses constant temperature anemometry techniques to measure bed shear stress. Two probes are mounted side-by-side for comparison. The bottom detector stops the T-bar from pushing the

shear stress sensors into the bed while the sensor mounting bracket allows the probes to settle on the bed by their own weight to obtain the most accurate measurement.

FLOW SENSORS

Under the same contract with the University of South Florida, metal-clad constant temperature flow magnitude sensors were developed. These sensors use constant temperature anemometry techniques for flow detection, but direction of flow is still under development. These sensors have far better resolution and fewer deployment problems than electromagnetic and impeller-type meters. They have one restriction in that they are temperature-dependent, so temperature is measured during deployment for calibration work upon retrieval. These flow sensors exhibit millimeter resolution which is needed in order to make measurements near the bed.

ELECTROMAGNETIC VELOCITY METERS

An array of five, 1/2-inch diameter electromagnetic velocity meters is included in the sensing capabilities of the EBIS system. They are mounted on the legs at any interval desired. The power requirements and data logging capabilities are provided by a separate data logger and power supply located at the surface. The Tattletale 6 is used to control these sensors because of the better bit resolution and larger mass storage capacity. These are primarily used for system verification during the development stages, but they also give the array the capability to make measurements of turbulence.

SOFTWARE

Software development was fairly easy to accomplish, but very high standards were maintained during this stage of development. A user-friendly software package was mandatory for the success of the EBIS system, since the system will be used by a wide variety of people with varying backgrounds. Checks were placed on every menu to prevent user error. The program was designed so that a highly technical manual was not needed to run the many different applications of the system. The data are easily downloaded into standard commercial spread sheet programs for editing and analysis.

DEPLOYMENTS

The EBIS system has been deployed five times at two different locations, Charleston Harbor, and Winyah Bay, South Carolina. Valuable information on deployment techniques and sensing ability of the system was learned from these deployments. The need to reduce system height and improve structural integrity for high flow deployments was identified from these deployments.

FUTURE DEVELOPMENTS

Further system modifications will concentrate on the reduction of structural drag and deployment techniques. The instrumentation package has reached a point where further modifications would only be to add additional sensors, conductivity and resistivity. Smaller deployment structures and instrument housings to use the developed sensing technology on will be an area of interest because one structure can not be used universally.

ACKNOWLEDGMENTS

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# SUSPENDED SEDIMENT MOVEMENT AND BOTTOM MATERIAL CHANGES IN A SMALL DYNAMIC ESTUARY

by

Timothy L. Fagerburg[1] and Allen M. Teeter[2]

## INTRODUCTION

Winyah Bay is an irregularly-shaped small tidal estuary extending about 16 miles from the ocean to the confluence of several rivers near Georgetown, South Carolina. The bay width is about 0.75 mile at the entrance between the North and South Islands, 4.5 miles in the middle section as it widens into a shallow expanse known as Mud Bay, and 1.25 miles in the upper section. Fresh water inflow into the bay is contributed by the Pee Dee, Waccamaw, Black, and Sampit Rivers. The total mean fresh water inflow into the bay is approximately 13,000 cfs.

The existing navigation project provides a 27-foot-deep channel from the Atlantic Ocean to the turning basin in the Sampit River near Georgetown. The route of the Atlantic Intracostal Waterway also passes through Winyah Bay. Over the past fourteen years, the material from annual maintenance of the channel has been disposed at locations along the northern side of the Western Channel Island in an attempt to create marsh lands for habitat development. However, the marsh area has been slow to develop. It appears that a larger percentage of the deposited dredged material is being eroded and possibly re-entering the system via the recirculation patterns within the bay.

This paper presents the data that were collected by means of a comprehensive field survey for the U. S. Army Engineer District, Charleston to attempt to quantify the transport pathways for the dispersed sediment material. The approach was to utilize long-term monitoring, short-term data collection during a tidal cycle and spot measurements of bottom material composition.

The short-term data collection effort was conducted to collect synoptic field data during a single tidal cycle within Winyah Bay. Data collected for longer term conditions covered the period prior to the start of dredging operations, during dredging, and for several weeks following the completion of the dredging. Measurements consisted of the following:

a. Current speed and direction at 3 ranges over one tide cycle;
b. Tide levels recorded at 3 locations;
c. Suspended sediment and salinity samples at each data collection range over one tidal cycle;
d. Automatic water sampler at 3 locations;
e. Core samples of bottom material at 13 locations formaterial classification.

## DATA COLLECTION EQUIPMENT

### Current Speed and Direction

Each boat used in the collection of data was equipped to deploy instruments over the side for measuring current speeds and directions. An indicator on the winch displays the depth of the instruments below the water surface. A Gurley Model 665 vertical-axis cup type impeller velocity meter with direct velocity read-out capabilities was used to measure the current speeds. Current directions are

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monitored with a magnetic directional indicator mounted above the velocity meter on a solid suspension bar. A streamlined lead weight holds the sensors in a vertical position and orients them into the direction of the flow.

Suspended Sediment and Salinity Water Samples

Water samples for analyses of salinities and total suspended solids were obtained at each depth that a velocity reading was taken by pumping the sample from the depth of the velocity meter to the surface collection point. The sample is pumped through a 1/4 inch inner-diameter plastic tubing. The opening of the sampling tubing is pointed into the flow. A portable pump is used to pump the water through the tubing to the deck of the boat where each sample is then collected in individual 8 ounce plastic bottles.

Automatic Water Samplers

Water samples were taken at regular intervals during the tidal cycle data collection effort and during the periods before and after the field data collection using ISCO Model 2700 automatic water samplers. Samples were collected in one-liter plastic bottles located inside the sampler. The samplers were programmed to collect four samples per bottle every 373 minutes during the peak flood and ebb tides and the slack water periods. Five samplers were used in this study to obtain water samples at various locations in Winyah Bay.

Tide Level Recorders

Water surface elevations for tide level determination were measured by Fisher-Porter Model 1550 punched paper tape mechanical water level recorders. These instruments record elevations to the nearest 0.01 foot and have a range of up to 100 feet. A timer activates the recording mechanism every 15 minutes, and the float elevation at the time is punched on 16-channel paper tape.

Bottom Sediment Samples

The samples of the bottom sediment were obtained using a push core-type sampler. The sampler consisted of a 1 1/2-inch diameter PVC pipe which was thrust into the bottom sediment at the sampling locations. Then, by means of a valve attached at the upper end, the pressure in the sampler pipe was reduced to hold the sample in the pipe as it was brought to the surface. The samples then were classified by visual inspection.

Procedures

For the one tidal cycle data collection period in the Winyah Bay study area, three ranges were selected at which data would be collected and are shown in Figure 1. Prior to the beginning of the data collection, the boats assigned to each range deployed anchors and mooring lines with inflated buoys at each of the stations. These mooring lines were used to hold the boat in position at each station for collection of the data. The velocity data and water samples were collected at three depths: near the bottom, mid-depth, and near the surface. The near-bottom measurement was made at a distance of 2 feet above the actual bottom. The mid-depth measurement was obtained at the actual mid-depth measurement. The surface measurement was obtained at a distance of 2 feet below the water surface. The data at each station were obtained once per hour.

Laboratory Analysis of Water Samples

The samples collected by the automatic water samplers and those obtained at the individual sampling stations during the survey were analyzed in the laboratory at the Waterways Experiment Station. Total suspended materials were determined by filtration of the samples. The salinities of each sample were measured using a Beckman salinometer with automatic temperature compensation. The salinometer was calibrated with standard seawater and was accurate to within ±0.2 ppt.
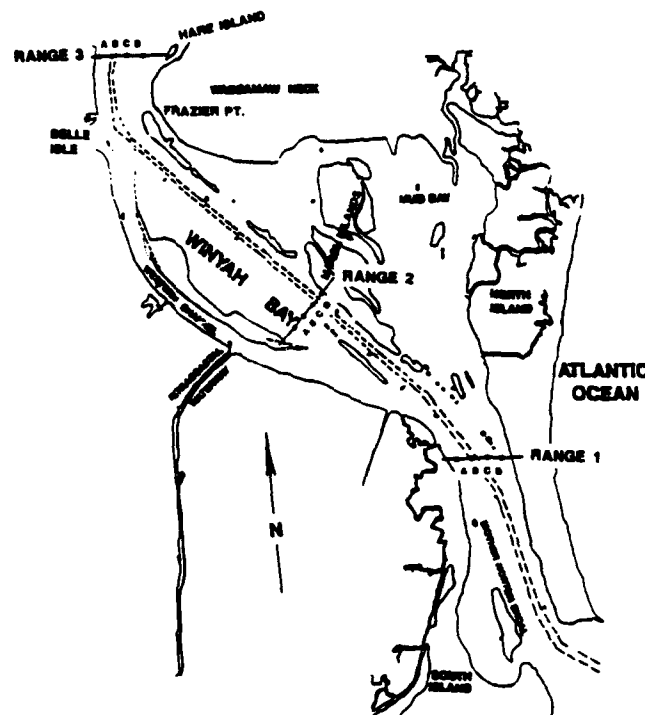
Figure 1. Location Map

## DATA PRESENTATION

### Tide Data

The tidal variation of the water surface elevation data wasobserved during the survey. Comparison of the data from the water level recorders was used to estimate tidal phase and range differences between the lower and the upper reaches of Winyah Bay. The maximum tide ranges were 4.63 feet and 4.48 feet at the lower and upper reaches of the bay, respectively. The tide phase difference was observed to be 0.50 hours between the lower and upper reaches of the bay.

### Velocity Data

A typical plot of the velocity data for each cycle of the tide (ebb and flood) during the survey period is shown in Figure 2. The maximum velocities were observed to occur near the surface and at the data station in the navigation channel. The maximum observed velocities were 5.5 fps, 3.4 fps, and 2.8 fps at Ranges 1, 2, and 3, respectively. No large amounts of fresh water inflow from the rivers local to this area contributed to the flow in the channel. As a result, no large variations, other than tidal, existed in the magnitude and direction of the currents. Eddies and unusual flow circulation patterns, other than those created by tidal effects, were not observed.

### Suspended Sediment Data

Figure 3 represents a typical plot of the suspended sediment concentrations observed during the tidal cycle data collection effort within Winyah Bay. The majority of the samples containing the greatest concentrations of suspended sediment were generally found near the bottom of the channel at the time of the strength of flood period. The suspended sediment concentrations near the bottom within the shallow areas tend to be lower than those observed in the channel during the peak velocity periods of the strengths of ebb and flood tide. However, a slight increase was seen in the surface concentrations within the shallow areas during the strengths of ebb and flood.

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Figure 2. Typical time history plot of the velocity data



Figure 3. Typical plot of the suspended sediment concentrations.

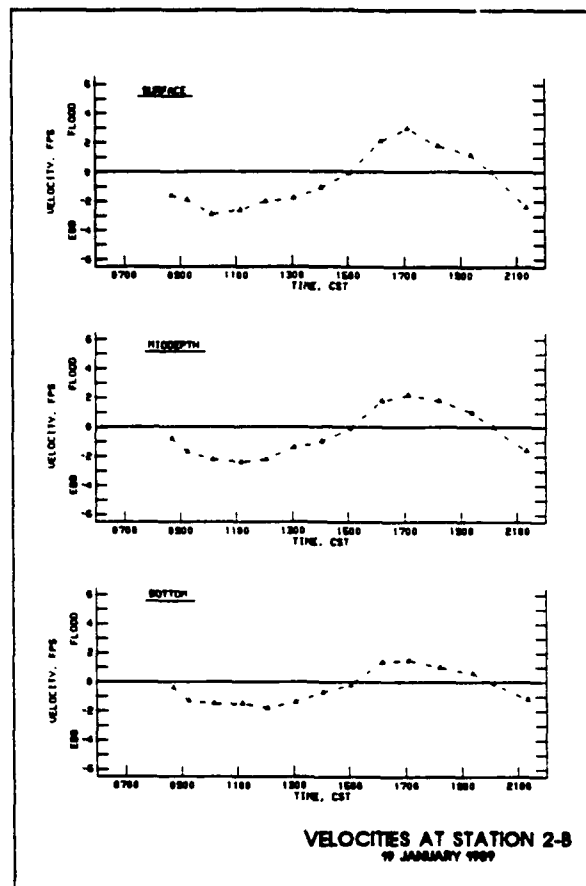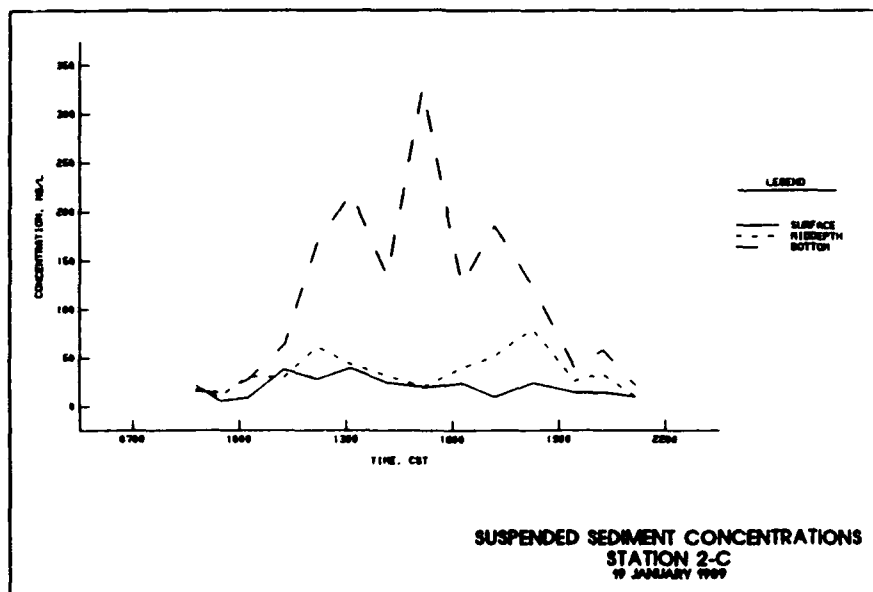Data sets from the survey were used to calculate net tidal fluxes and flux components for suspended solids. Station data consisted of 14 hourly samplings at 3 depths, with the exception of one station (2A) which was omitted from this analysis. Temporal coverage was greater than a tidal period, and the data sets were trimmed to include roughly the 0800 to 1900 hour period of 19 January 1989.

Large net flows were indicated by the trimmed data sets, indicating a tidal bias in the ebb direction or the effects of freshwater inflow to the estuary. This was especially true at the upper bay data collection range (Range 1). There were insufficient data with which to check the salt balance of the estuary as a way of determining whether the trimmed data set was representative of a typical tidal cycle.

The results of the tidal suspended sediment analysis indicated that net fluxes were all seaward, not typical of other estuarine areas where net fluxes are frequently landward. As noted above, maximum concentrations occurred during the flood tidal phase. Removing the net-flow flux component from the total fluxes still yielded net fluxes which were seaward with the exception of one station (3C), which showed a small landward net flux.

Seaward fluxes were greatest south and west of the navigation channel. While the net-flow flux component was largest at 5 stations, the flux component caused by vertical deviations in fluctuations was also greatest at 5 stations.

It was observed that suspended sediment concentrations are dependent upon the surface wave conditions within the bay, particularly in the shallow areas. The suspended sediment concentrations correlated well with the tidal effects resulting from spring-neap variability of tides as indicated in Figure 4. The spring tides will transport much greater quantities of suspended material than will neap tides and will be critical to flushing characteristics of the estuary.

The results of the suspended sediment concentration analyses performed on samples obtained during the pre-dredging period were found to vary from 140 mg/l at the lower portion of the bay to a minimum of 6 mg/l at the upper portion. The analysis of the samples obtained during the dredging operations revealed a maximum suspended sediment concentration of 86 mg/l. From the samples obtained immediately following dredging, the analysis yielded a maximum concentration of 103 mg/l. The data do not indicate any significant increases in suspended sediment concentrations resulting from dredging operation.

Bottom Sediment Classification

The bottom sediment samples collected by means of the push core sampler were classified by visual inspection in the field   A typical example of the results of the visual inspection of the samples are shown graphically in Figure 5.  The samples shown in Figures 5a and 5b were obtained prior to the start of and immediately following completion of dredging operations, respectively.  This analyses indicated some differences in the bottom sediment composition at most of the sampling locations occurring over a 5-month period.  In general, these changes reflect the movement of the very fine bottom materials out of the sample areas by the natural circulation of the bay resulting from tidal forces.  These changes may also reflect variations in relocation of the sample point during the two separate sampling periods and to the discrepancies that are inherent to this type of sampling technique.
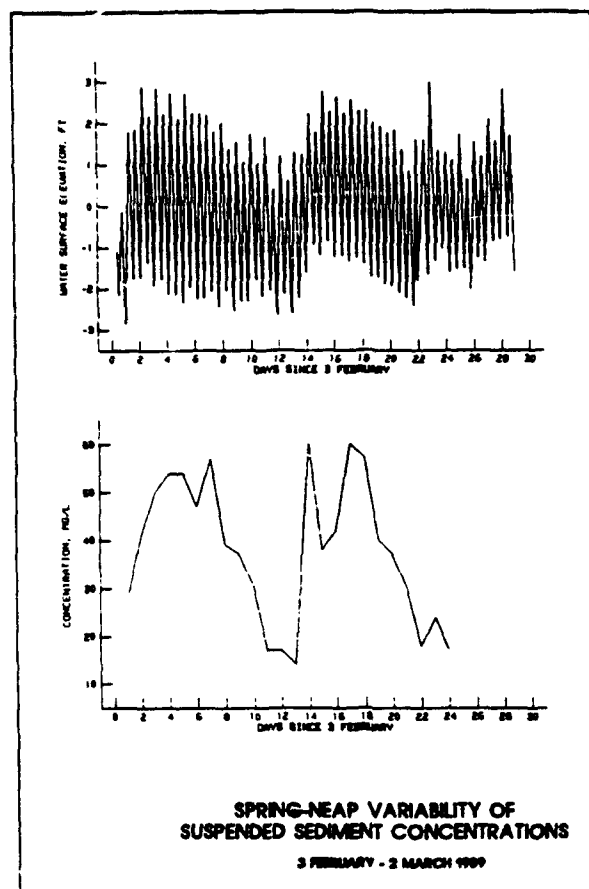
Figure 4. Spring - neap variability of tides and the effect on the suspended sediment concentrations.
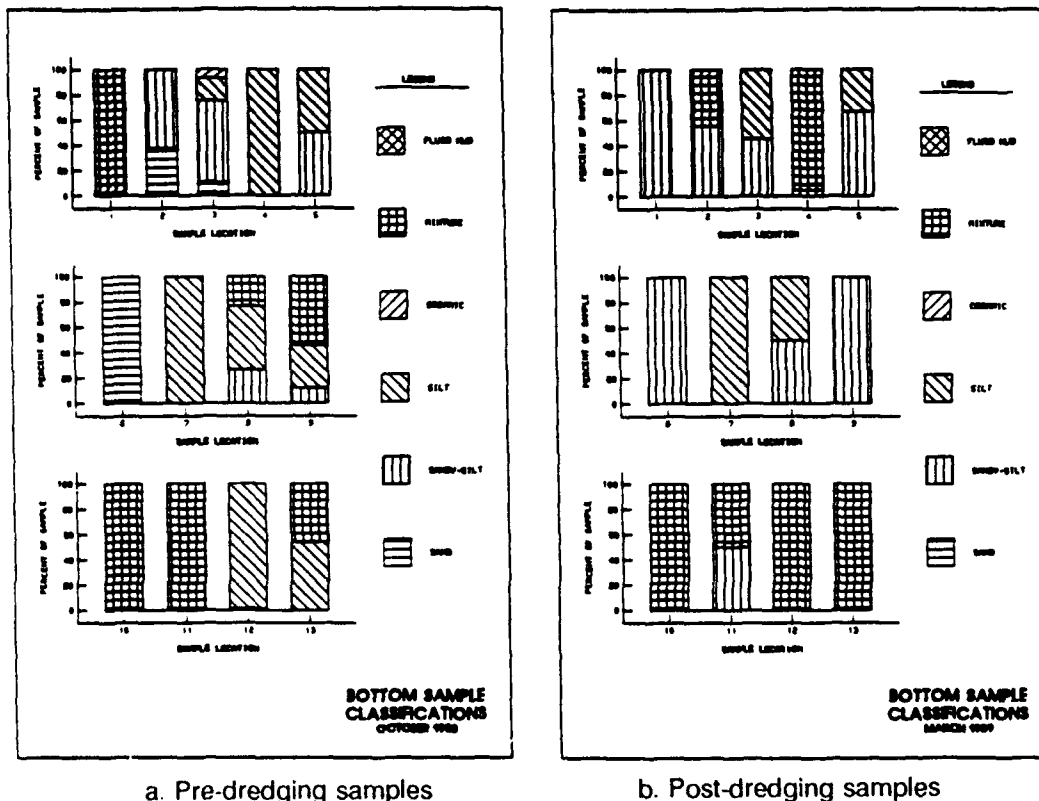


a. Pre-dredging samples          b. Post-dredging samples

Figure 5. Graphical representation of the bottom material classification by visual interpretation.

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## SUMMARY

The data presented herein were collected from the intensive survey and longer-term sampling efforts within Winyah Bay. The following observations were made of the data:

a. There appears to be a slight decrease in the maximum range of water surface elevation (tide) (0.15 foot) from the lower bay.

b. The maximum velocities observed during the survey occurred at the strength of ebb of the tidal cycle. The maximum recorded velocity was 5.5 fps.

c. Suspended sediment concentrations in the channel were found to be generally greater near the bottom. The greatest suspended sediment concentration observed during the survey period was 420 mg/l.

d. Net tidal fluxes of suspended sediment were large and were in the seaward direction.

e. Long-term sampling of the suspended sediment recorded a maximum concentration of 140 mg/l. This concentration occurred during the pre-dredging period. The long- p73term suspended sediment sampling indicated no significant changes in concentrations resulting from dredging operations.

f. Changes in the bottom material composition over time indicates the most of the very fine sediments are not retained within the majority of the sampling areas but appear to be displaced by the natural circulation patterns within the bay.

g. Salinity values indicated that the lower portion of the bay could be described as being well-mixed, while the upper portions could be described as being partly- to well-mixed.

## ACKNOWLEDGMENTS

# MODELING OF EFFLUENT WATER QUALITY FROM CONFINED DISPOSAL OF POLLUTED DREDGED MATERIALS

by

Earl J. Kenzie, P.E.[1]

## INTRODUCTION

The proposed dredging of polluted sediments from Indiana Harbor and Canal required the investigation of several alternatives for confined disposal. The confined disposal facilities (CDFs) considered include an in-lake structure and three upland containment areas. In order to predict the quality of effluent from these CDFs, a water quality model was developed and run.

The water quality model utilized input of several parameters which are dependent upon the operation of the CDF, including: the site-specific layout of the containment structure; the method of disposal (hydraulic, mechanical) and the physical arrangement of dredged materials within the CDF; the in-situ pore water quality of the sediment; and the percentage of pore water released from both saturated and unsaturated sediments during placement and rehandling within the CDF. Physical soil properties of the dredged sediments were utilized to quantify the pore water releases. Nitrification of ammonia nitrogen was simulated in the model, including the effects of temperature on the nitrification rate.

## MODEL OVERVIEW

The simulation of CDF water quality must account for a number of physical and chemical processes which may occur inside the CDF, including: settling of solids; delta formation; changes to the CDF pond volume; release of soluble contaminants during disposal; dilution of dissolved contaminants in pond water; oxidation/reduction of contaminants; and *dewatering/consolidation of sediments*. The first part of the water quality model is a mathematical representation of the physical environment of the empty CDF (surface areas, depths, and volumes). The above mentioned processes occur within this physical environment and respond to changes in one another.

The simulation of water quality for an in-lake CDF is more complicated than for an upland CDF, due to the presence of a large pond which is a permanent feature for nearly the entire life of the facility. An upland facility may have large or small ponded areas during individual disposal operations. The model for the three upland CDFs is basically the same *mathematical representation*; thus, the following discussion is separated into the two types of facilities considered (in-lake and upland CDFs).

## IN-LAKE CDF WATER QUALITY MODEL

The in-lake CDF model simulated the concentrations of dissolved and particulate contaminants in the CDF pond in a time series analysis. The underlying assumption is that the CDF pond functions as a fully mixed reactor. The in-lake CDF model is composed of two parts. The first is the model of the dredged material delta formed as the facility is filled. This is purely a representation of the physical phenomena. The second part is the mass balance analysis which includes the release of soluble contaminants to the CDF pond, dilution in ambient pond water, and chemical transformation of selected contaminants.

The most complicated part of developing a model to simulate the filling of a CDF is how to mathematically represent the physical processes which take place inside the facility (i.e., an accounting of the dredgings and water). In order to develop these accountings, it is necessary to know how the

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CDF will be operated. This includes the methods of dredging and disposal, rate of disposal, location of placement, and operational controls. The model must be able to represent not only how fast the dredgings enter the CDF, but also where the dredgings lie relative to the water surface.

The delta volume algorithms were developed based on the interior of the CDF being represented as a trapezoidal prism. The dredged material delta was depicted as a trapezoid. There are three cells within the in-lake CDF and the model represented the filling of this facility in four operations for each cell, or a total of twelve operations. Since access to the site for disposal operations is possible only from three sides of the adjacent cells, it was determined that reworking of the dredged sediments will be necessary.

The amount of material both above and below the pond water was quantified by applying the delta volume algorithms within the physical restraints as determined by the filling operations. This information is needed to be able to account for contaminant loadings from saturated and unsaturated dredged sediments. Sensitivity checks were incorporated into the delta formation model based on experience and best available information to assess the reliability of the simulated values.

The analysis for the in-lake model utilized volume-weighted bulk chemical concentrations and averaged sediment pore water concentrations based on test results from the in-place sediments. These pore water and bulk chemical concentrations were assumed constant for all parameters except PCB, which varied depending upon whether the sediments being dredged were classified as backlog TSCA or non-TSCA materials, or as maintenance materials. The concentration of PCB associated with each type of sediment was determined by calculating a weighted average from the results of several previous sampling and analysis events. For the TSCA sediment, the weighted average PCB concentration was calculated to be 38 mg/kg; for the non-TSCA sediment, the level was calculated to be 6 mg/kg; for the future maintenance sediment, the concentration was calculated to be 2 mg/kg. The maintenance material PCB concentration was determined from analysis results of surficial grab samples taken in 1987, 1988, and 1989.

Information concerning the quality of pore water was obtained from the results of laboratory analyses performed by the Waterways Experiment Station (WES) on Indiana Harbor samples (USACE, 1987). Direct analysis of pore water leached from sediments was conducted for some chemical parameters (USACE, 1987). For other parameters, the pore water concentrations were back calculated using the results of a modified elutriate test on sediment samples (USACE, 1987). This back calculation was performed based on the fact that the modified elutriate test (Palermo et al, 1986) combines one volume of sediment with four volumes of site water and that in-situ sediment is approximately 2/3 pore water by volume. This method of pore water concentration estimation is extremely conservative. It can greatly overestimate the pore water concentrations for those parameters which are desorbed from sediment particles during the elutriate mixing procedure.

The concentration of PCB in the sediment pore water was determined by using equilibrium partitioning concepts. The distribution coefficient (Kd) for PCB was measured through laboratory analyses performed on Indiana Harbor sediment collected in 1985 (USACE, 1987). This distribution coefficient was applied to the weighted average PCB concentration of the sediment to determine representative dissolved PCB concentrations in the pore water for TSCA (0.148 ug/l), non-TSCA (0.023 ug/l), and future maintenance sediments (0.008 ug/l).

For some chemical parameters no elutriate test results or pore water analyses were available. The concentrations used for these parameters were estimated using data from other contaminated sediments or previous modeling efforts.

An in-lake CDF is a piece of lake that has been surrounded by a stone filled dike. Monitoring data from the Chicago Area CDF indicates that the water quality of the CDF pond before disposal operations have begun, and for the majority of time between disposal operations, is essentially the same as the adjacent lake (USACE, 1985a, 1986a, 1987a). The initial water quality of the in-lake CDF was assumed comparable to nearshore Lake Michigan, and recent monitoring data was used (IDEM, 1986).

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Ambient levels of PCBs were assumed to be 5 ppt. This concentration is considered representative of ambient conditions in nearshore southern Lake Michigan. Most sampling and analysis programs have detection limits that exceed this level for PCBs or do not test for this parameter at all.

It was reasoned that during the disposal of mechanically dredged sediments to an in-lake CDF, soluble contaminants will be released to the pond in two methods. The first method is the release of sediment interstitial water to the pond as sediments are placed into the water and settle rapidly to the bottom. The second method is the drainage of interstitial water from sediments that are placed on the delta above the waterline. To model the water quality of the CDF pond, one must determine how much of the sediment pore water is released and drains from the sediments during disposal. Once the sediments are placed and have settled to the bottom or have drained available excess water, they no longer release significant loadings to the pond (relative to the loadings from actively disposed sediment).

The amount of pore water on a volume percentage basis in the sediments was calculated by using the percent solids (by weight) and specific gravity of the sediments. In order to estimate how much of this pore water is available for release to the pond, the amount of saturated and unsaturated sediments was determined from the delta formation model. For operations 1, 2, and 3, it was assumed that approximately 97% of the dredgings would be submerged. For operation 4, about one-half of the dredgings was assumed to be submerged.

The release of pore water from sediments which are placed above the waterline is estimated from the physical and engineering properties of the sediments. Mechanically dredged sediments typically have the same solids and water contents as in-situ sediments. This has been verified from samples of sediments collected at the Chicago Area CDF during disposal (USACE, 1985a, 1986a, 1987a). Dredged material placed on an upland surface will lose water by drainage and drying. Fine-grained dredged materials will, after a long period, typically have a water content approaching its liquid limit (USACE, 1987). As a conservative assumption, the percent of pore water available for release by drainage from unsaturated sediments was determined as the difference between the sediment water content as disposed and the liquid limit.

The release of pore water from sediments disposed directly to the CDF pond are determined by physical factors. Sediments that are disposed into the CDF with significant force or which settle through significant depths should have greater opportunity for pore water release than those which slide into a delta which has mounded to a height near the waterline. No laboratory tests have yet been developed to quantify the percent release factor for saturated sediments. For the application of this model, dredgings which travel more than 10 feet from the pond water surface before reaching the base of the CDF were considered to experience 100% release of the sediment pore water. Disposed sediments which traveled 10 feet or less before contacting the CDF bottom were considered to undergo a 90%-10% release of sediment pore water, depending upon the underwater travel distance.

The withdrawals from the CDF were determined based on the volume of CDF pond water displaced during each week of the various operations. The amount of water pumped from the CDF is equal to this CDF pond water displacement volume. Thus, the pond volume is constantly changing on a weekly time step according to the above loading and withdrawal information. The dissolved concentration of a chemical contaminant in the CDF at a given time was calculated through a mass balance equation which utilizes the loading and withdrawal information and the pore water concentration for the chemical parameter of interest. This mass balance equation is based on the assumption that the volume of pore water released from saturated sediments is replaced with pond water having a quality equal to the dissolved chemical concentration from the previous week.

Most chemical parameters were assumed to be conservative by the in-lake model. As a result, the output from this model overestimates the concentrations of dissolved chemical substances which will occur under field conditions. The only parameters for which chemical or biological reactions were simulated are ammonia nitrogen and PCB. Nitrification was simulated using a first-order reaction equation. For this application of the model, nitrification rates were obtained from literature values (USEPA, 1985). A nitrification rate (K20) of 0.1/day and a nitrification constant (A) of 1.08 were

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considered representative of the freshwater system being modeled. These nitrification values were used with the water quality model of the Chicago Area CDF and were found to produce results consistent with CDF pond-monitoring data. The water temperature was differentiated on a weekly basis and was taken from monitoring data for the nearshore lake.

The in-lake model calculated the concentration of PCB in the CDF pond by using two approaches. The first approach applied the mass balance algorithm to the pore water and pond water concentrations as described earlier. The second approach used algorithms developed by the U.S. Environmental Protection Agency (USEPA) Environmental Research Laboratory in Athens, Georgia. The USEPA laboratory developed a spreadsheet model for application to CDFs at the request of USEPA Region 5. This model determines the dissolved level of PCB in the CDF pond through equilibrium partitioning concepts, specifically by assuming equilibrium with the PCB associated with suspended solids.

The suspended solids concentrations used for the in-lake model were derived entirely from the operating and monitoring experience at other CDFs where mechanical dredged disposal was used. The suspended concentration of a chemical constituent was estimated using the suspended solids concentration and sediment bulk chemical concentration.

## UPLAND CDF WATER QUALITY MODEL

The water quality impacts for the upland CDFs were simulated by using two different model approaches, one for mechanical disposal and the other for hydraulic disposal. For the mechanical disposal operations, the Hydrologic Evaluation of Landfill Performance (HELP) model, developed by WES for USEPA, was applied. The model was designed to evaluate barrier systems and other environmental controls used at landfills (Schroeder, 1984). For the hydraulic disposal operations, a variation of the in-lake CDF model was applied. This model was much simpler than the in-lake model. There were no delta formation algorithms with this model, and the CDF pond was considered to be a transient feature.

Mechanically dredged material placed on an upland surface will lose water by drainage and drying. Although localized ponding may occur for limited times during disposal or after storms, there is no permanent pond. The drying of dredged materials and formation of a crust is a two-stage process. First-stage drying typically results in the dredged materials having a water content equal to 1.8 times their liquid limit (USACE, 1987). Second-stage drying continues until a water content of about 1.2 times the plastic limit.

The HELP model was applied to develop a water budget for the upland CDF with mechanical disposal. This was done because mechanically disposed materials have so little free-draining water that other factors, such as precipitation and evaporation/transpiration were considered more influential. The HELP model is a hydrologic model which can predict the amount of water which falls onto a surface, the amount that runs off, the amount that percolates into the ground, the amount lost through evapotranspiration, and the amount collected by leachate collection systems.

The upland CDFs will have an underdrain, comparable in some ways to a leachate collection system. Any water which drains from the dredgings, as well as precipitation, will be collected by this system and pumped from the CDF. The quantity and quality of this pumped leachate were predicted by utilizing the HELP model for a number of CDF surface conditions, as listed below:

Empty - post construction CDF floor with no dredgings placed
New - dredged material surface during or shortly after disposal (no vegetation)
Aged - dredged material surface some time after disposal, with moderate to dense vegetation
Capped - surface of CDF post cap/cover placement

One or more of these conditions may occur within the upland CDF at one time. The total surface area of the CDF at a given stage of filling was represented by some combination of these conditions. For

example, at the half-filled point, one third of the CDF surface area would be capped, one third would be empty, and the remaining third would be divided between new and aged.

The suspended solids level in the water pumped from the underdrain system was assumed to be around 50 ppm. The underdrain will either be trenched into existing surface soils, composed of fine-grained sand or be constructed with a layer of sand/gravel from offsite. The presence of the sand layer will restrict the movement of solids into the underdrain, much like a filter.

Hydraulic disposal of mechanically dredged sediments was an option investigated for two of the upland CDF sites. Under this method, barges of materials would be brought to the unloading area and the sediments slurred by the addition of canal water and piped to the CDF. The amount of additional water required to slurry and transport the dredged materials is dependent upon several factors; sediment physical characteristics, pump type, pipeline distance, etc. The amount of additional water needed to transport the dredged material is very important to the water quality model since it represents an additional volume of water to be drained, treated, and discharged.

The volume of the pond formed during hydraulic disposal is a function of the rates of pumping both into and out of the upland CDF. The pumping rates are determined by the dredging rate, slurry composition, and detention time required for adequate settling. For this model application, the quantity of additional water necessary to hydraulically dispose of the sediments was assumed to be four times the volume of the dredgings. This 4:1 ratio is representative of a standard cutterhead hydraulic dredge, and is also the ratio used for the elutriate test procedure. This 4:1 (additional water:sediment) ratio used was considered to be the worst-case scenario.

The pond volume was expressed on a weekly basis for the hydraulic disposal method. This volume is a function of the pore water released from the sediments, the volume of the slurry water used, the volume of the pond from the previous week's operations, and the volume of water pumped from the CDF. The quality of water drained from the upland CDF during hydraulic disposal for dissolved contaminants was determined from the modified elutriate test results as performed in the laboratory.

The level of suspended solids in the water was determined through settling tests using several flocculants (USACE, 1980, 1987). With extended settling times, followed by polymer flocculation and secondary settling, the water drained from the upland CDF was considered to contain suspended solids levels of 50 ppm or less. This suspended solids concentration and bulk chemical levels of the sediment were used to determine the level of particulate contaminants in the upland CDF water, similar to the approach used for the in-lake model.

## RESULTS

The in-lake CDF water quality is much better during the earlier operations for each cell and for the first two cells filled as compared to the last cell filled. This trend is largely due to the effects of dilution prior to discharge. Treatment of the effluent will result in compliance with state standards for most chemical parameters.Contaminants not in compliance will achieve ambient lake concentrations within a small mixing zone. The highest level of treatment required to satisfy state standards is for effluent from the final disposal operation (i.e., operation 4 for Cell #3).

As a basis of comparison, mechanical disposal to an upland CDF appears to produce the lowest concentration for most parameters and has the least flow of any disposal option. The flow decreases significantly as the filling operations progress, and the concentration of contaminants increases due to less dilution from seepage of rainwater from adjacent cells.

Comparison of mechanical and hydraulic disposal shows that during the initial filling operations, the water quality of the untreated effluent is less for all parameters with mechanical disposal. However, during the final filling stages, the concentrations of ammonia nitrogen, TKN, phenols, and cyanide would be higher for mechanical disposal.

Hydraulic disposal methods may be utilized which require less additional water to transport the sediments than the volumes assumed for the upland CDF model. The results discussed here reflect an operation using a 4:1 (added water:sediment) volume ratio. The less water added, the more similar the water quality and quantity results of the hydraulic and mechanical disposal methods become.

## CONCLUSIONS

The models applied to the in-lake and upland CDFs relied upon several assumptions concerning the concentrations of contaminants in the sediment pore water and the relative volumes of this pore water released during disposal operations. Direct analysis of the pore water concentration for each contaminant of interest would reduce the uncertainty of back calculations from other types of tests or deriving information from previous modeling efforts. Development of a laboratory simulation for determining likely volumes of pore water releases during disposal operations would also increase the reliability of the model predictions.

The physical processes which take place inside the CDF during filling and rehandling must be accurately described to adequately assess water quality. As much information as possible about the physical aspects of the CDF should be obtained before any modeling task is undertaken. This information is often dependent upon the dredging contractor's discretion and cannot always be specified to be consistent with the assumptions of the model.

The models presented here produced results which are conservative. There was no simulation of chemical reactions which are likely to take place within the CDF, such as precipitation and oxidation/reduction. Development of algorithms to simulate these processes would improve the accuracy of the model results.

The mass balance models for the in-lake and upland CDFs accounted for much of the contaminant losses from the disposed sediments. However, there is a significant loss of PCB through volatilization of this contaminant during dredging and disposal. Another model was developed at the Chicago District to assess this volatilization loss and is presented as a separate paper by Mr. Jay Semmler for this conference.

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# DROUGHT INDUCED SALTWATER INTRUSION ON THE MISSISSIPPI RIVER

by

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and Burnell J. Thibodeaux[3]

## INTRODUCTION

The Mississippi River debouches into the Gulf of Mexico through three major distributary channels some 100 miles down river from the city of New Orleans, La. From the point of major trifurcation to a point some 240 miles above the mouth, the depth is 55 feet or greater, except for a few short reaches aggregating no more than 20 miles located above New Orleans. In order to provide deep draft access to the ports of New Orleans and Baton Rouge, La., the largest distributary, Southwest Pass, must be dredged on a regular basis from the trifurcation some 20 miles to the Gulf of Mexico. A joint venture by state and Federal agencies to increase the draft, below New Orleans, from 40 to 45 feet was completed in 1987. This required only the dredging of Southwest Pass.

Historical records of saltwater intrusion into the Mississippi River indicate that river discharges of 300,000 cfs (cubic feet per second) or greater are capable of preventing intrusion into the river at the mouth of Southwest Pass. A discharge of 250,000 cfs will allow saltwater to advance into the river to the point of major trifurcation, commonly referred to as the Head of Passes. All distances are commonly referred to with respect to miles AHP (Above Head of Passes).

The saltwater in the Gulf is about 2% denser than fresh water and it intrudes along the river bed at lesser flows. For example, 160,000 cfs will allow intrusion to advance to 55 miles AHP. Although the dominant factor is the volume of flow in the river, other factors that can influence the movement of saltwater include flow duration, channel slope, wind velocity and direction, tides, channel depth, and water temperature. A plot of the annual upstream excursions of saltwater versus the minimum river discharge, shown on Figure 1, demonstrates graphically the relationship between the location of the saltwater and the seasonally low flow in the river. Although flow duration is not factored into this plot, it serves as a useful tool in forecasting the probable location of the saltwater. For example, a river flow of 120,000 cfs would allow the saltwater to move upriver, along the bed to New Orleans, a distance of more than 100 miles.

## ESTUARINE HYDRAULICS

In estuaries, water of variable density causes differences in the magnitudes, distributions, and durations of currents, as compared to those of a single density system (Shultz and Simmons,1957). Where the mixing of freshwater and saltwater is slight, as is the case in Southwest Pass, the transition from freshwater to saltwater is sharply defined, and occurs over only a few feet of depth. The well defined line of demarcation is commonly referred to as the interface.

The degree of mixing that takes place in an estuary is the key to the solution to estuarine problems. In the Mississippi River and Southwest Pass, the degree of mixing is classified as a classic example of a highly stratified estuary. In such estuaries, the magnitudes, durations, and directions of the downstream currents which occur above the interface are appreciably different from those in the saltwater below, as shown in Figure 2.

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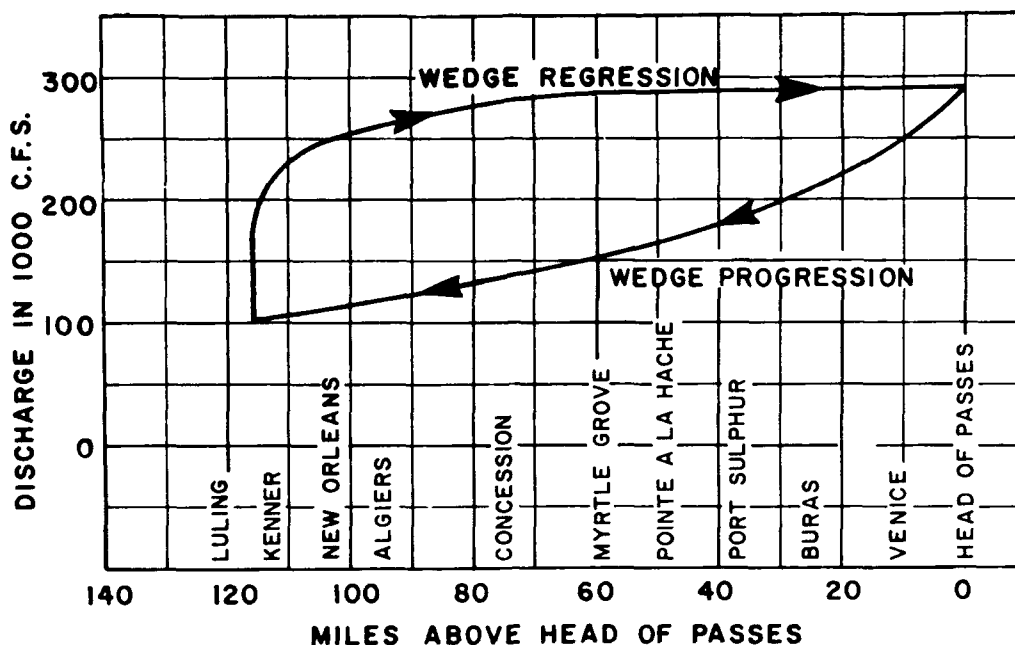
Figure 1. Position of Saltwater Wedge for River Flows Less Than 300,000 cfs.
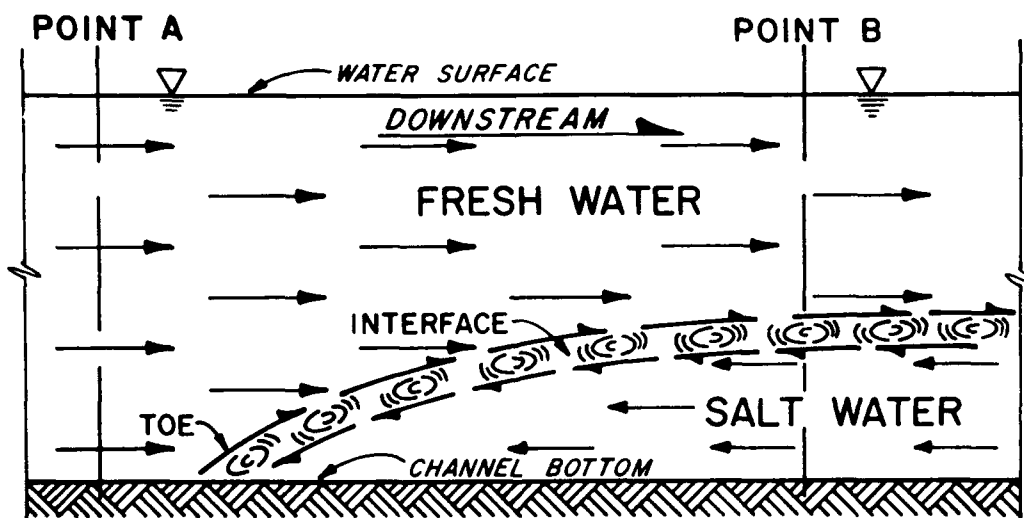


Figure 2. Typical Characteristics in a Highly Stratified Estuary.

At point A (in Figure 2), the direction of flow throughout the entire depth is toward the sea at all times, with a vertical velocity distribution similar to that of an upland stream. At point B, farther downstream, the flow in the fresh zone is still towards the sea, but the direction of flow in the underlying salt zone is upstream at all times. The constant upstream flow of saltwater below the interface, even when the location of the wedge-shaped interface is stable, is due to the constant erosion of the saltwater from the interface by the turbulent, outflowing freshwater which competes for the available channel cross-section. The point where the interface (5000 ppm Cl') intersects the river bed is commonly referred to as the "Toe" of the wedge. The saltwater thus eroded through the interface must be

constantly replaced or, in time, the wedge would be swept out to the Gulf. Downstream-flowing fresh-water velocities are a maximum near the water surface and at the entrance to the estuary. Upstream-flowing saltwater velocities are a maximum below the interface above the river bed. A point of current reversal is found at the interface where saltwater is diffused vertically into the freshwater and is returned to the sea in a highly diluted state.

FIELD LOCATING THE WEDGE

The interface between the two densities may be located in the field in several ways. The New Orleans District does this by dropping a conductivity probe down through the water column while making note of the point where there is a sudden and sharp increase in conductivity. This can range from about 1,000 micromhos ( 1 ppt salinity), just above the interface, to 16,000 micromhos (9 ppt salinity) at the interface in less than a 10-foot increment of depth. In order to adequately define the interface and the toe, several salinity profiles are taken along the river. A plot of three salinity profiles taken to establish the location of the toe during the 1988 intrusion event is shown in Figure 3.
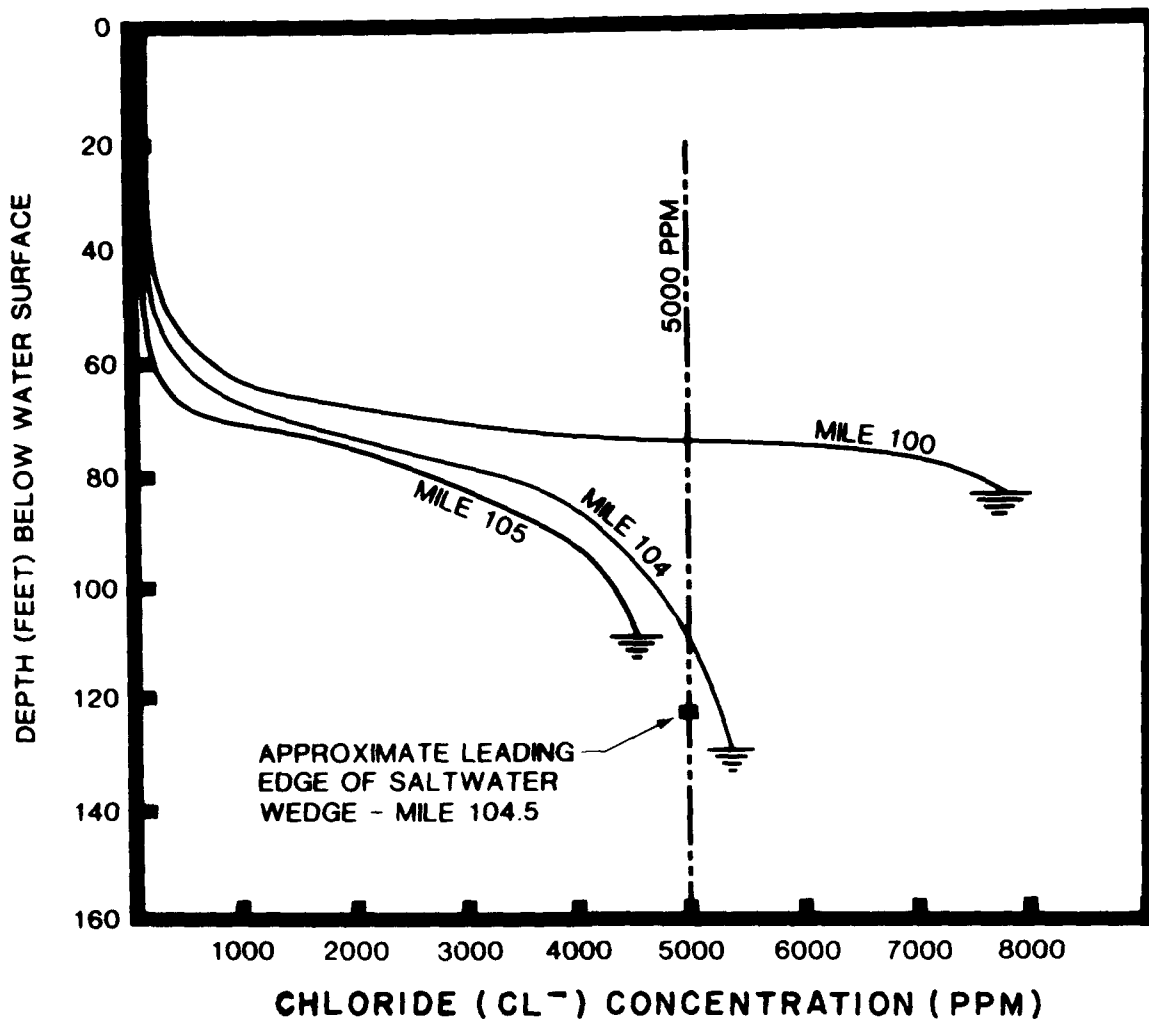
Figure 3. Locating the Toe of the Saltwater Wedge.

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The intersection of the 5,000 ppm Cl⁻ line with the profile establishes the depth of the interface at that point along the river on any given date. Successive daily plots can be used to track the movement of the toe of the wedge as it advances or recedes along the river bottom. The presence of saltwater at the river surface is usually not found in significant levels immediately above the toe.

RECORD DROUGHTS ON THE MISSISSIPPI RIVER

For the period of record, 1930 to 1988, there have been 21 occurrences of minimal annual daily low-flow in the month of October; 18 in September; 12 in November; and 5 in August. The 1988 drought was the first event to produce a minimal annual daily flow in July. There were eight occurrences of minimum average monthly flows; the six worst years are shown in Figure 4. The 1988 event produced the lowest average monthly values for May, June, and July, and was second only to the 1936 drought in August and September. The 1988 event had the longest low-flow duration of record.
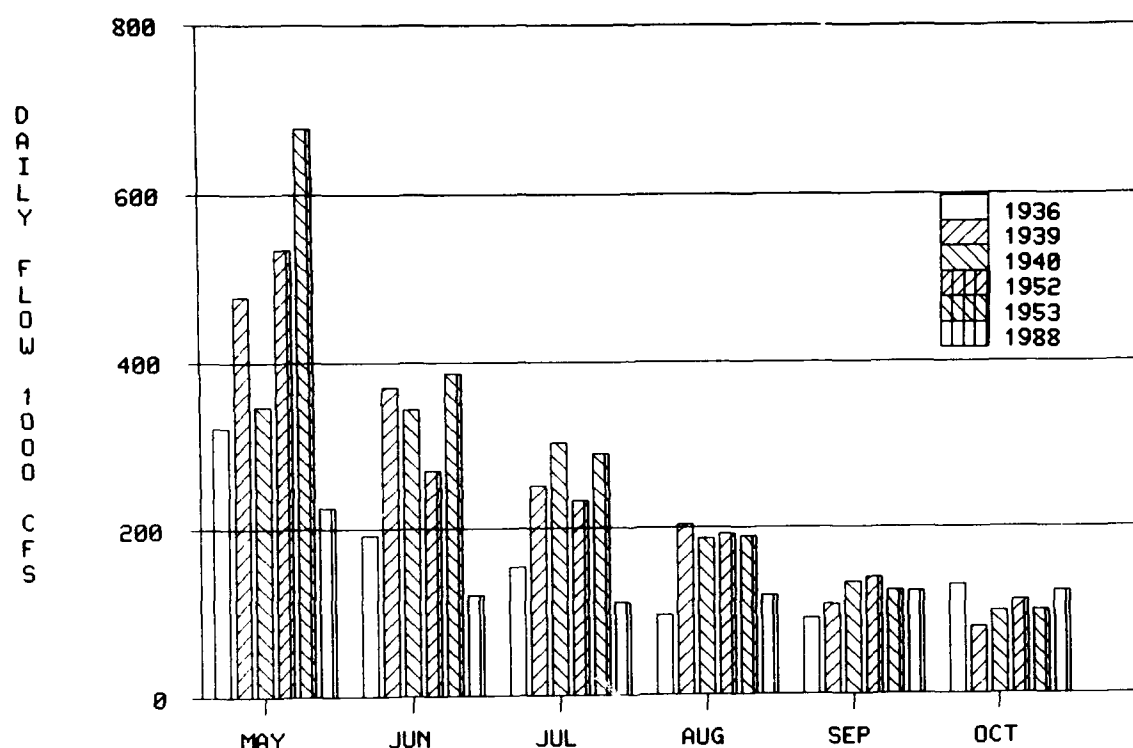


Figure 4. Comparison of Lowest Monthly Average Flows for 1930 to 1988.

IMPACTS ON WATER QUALITY

The minimum salinity standard for drinking water has been set by the Environmental Protection Agency (EPA) at 250 ppm Cl⁻. Since 1930, municipal and industrial water users in the New Orleans area had to cope with a substandard water quality on seven notable occasions, the most severe one having been in 1936 when salinity was greater than this standard for 32 days. Down river, at the community of

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Pointe-a-la-Hache, mile 49 AHP, salinity exceeded the standard for 66 days. By comparison, the standard was exceeded for 122 days at the same community in 1988; easily the worst event of record.

Table 1 shows the years of drought, in column (1); the peak daily salinity level at a New Orleans intake at 96 miles AHP during each drought, in column (2); and in column (3), the most upstream point reached by the wedge toe.

TABLE 1
Years When River Water Exceeded EPA Standards

| YEAR | SALINITY | LOCATION AHP |
|------|----------|--------------|
| 1930 | 320 | No Record |
| 1936 | 620 | <116 |
| 1939 | 463 | 119 |
| 1940 | 424 | 119 |
| 1952 | 341 | 116 |
| 1953 | 380 | 116 |
| 1956 | 250 | 116 |

## PROBLEM DESCRIPTION

There are 13 municipal and 15 industrial water supply facilities in the metropolitan New Orleans area which draw 223 and 245 mgd (million gallons per day) from the river, respectively. There are more than 1.2 million people who depend on the river as the only acceptable source in the entire region. It has been shown that it is impractical to transport the required volume by pipeline from suitable points upriver and other sources are of unacceptable quality due to hardness, odor, salts, or color, such as locally available ground water. It is practical, however, on a small scale to transport fresh water to the smaller users (<3mgd) below the point of intrusion by barging fresh water from a point upstream of the contamination. These smaller users include river communities of less than 5 thousand people and oil refineries located downstream of New Orleans.

## PROJECT FORMULATION AND IMPLEMENTATION

Until 1987, the Mississippi River and Southwest Pass could accommodate ocean going vessels drawing 40 feet below mean low water. A subtantial body of historical evidence pointed to channel deepening as the major cause of increases in frequency and duration of saltwater intrusion events. Consequently, when a depth of 55 feet below mean low water was authorized in 1985, it was appropriate that any design studies include engineering measures that would be capable of mitigating the definite impact on saltwater intrusion. The original design criteria had as its goal that water supplies along 120 miles of the lower Mississippi River not be degraded as a result of the channel deepening, and that this be accomplished without adverse impacts on navigation or flood control improvements.

Such criteria quickly ruled out locks or gated structures. The most acceptable plan involved the construction of an underwater embankment at a strategic location on the river bed at such height as to create a large resevoir which would store intruding saltwater for a period of days equal to the increase in duration caused by the deepening, and the barging of supplemental water to users below the sill.

In order to arrive at a satisfactory design height and location to place this sill, extensive mathematical modeling was conducted at the Waterways Experiment Station, Vicksburg, MS. The modeling effort for design has been reported by Johnson, Boyd, and Keulegan, 1987. It was later

determined by the State of Louisiana and the Federal government that the initial channel deepening should be to 45 feet below mean low water. As a direct consequence, additional modeling was necessary to determine the best height for the sill since the initial depth was 10 feet deeper. Hydrologic and hydrographic data collected in 1981 were used to calibrate the model known by the acronym LAEM (laterally averaged estuarine model). The model was verified to historical intrusion data, using an experienced discharge hydrograph as the upstream boundary condition for three known events. The effects on duration of intrusion were then computed for a deeper channel, with and without a submerged sill of several elevations and locations to determine the best overall plan. Figure 5 is a plot of the duration of saltwater intrusion for a repeat of the 1953-54 hydrograph for pre-project and for project, with and without a sill, at elevation -60 feet at mile 63 AHP. The results demonstrate that the sill has a marked stablizing impact on saltwater intrusion above the location of the sill; in fact, the sill case with the 45-foot channel produced an intrusion event of lesser duration than the 40-foot channel at all locations upstream of the sill.
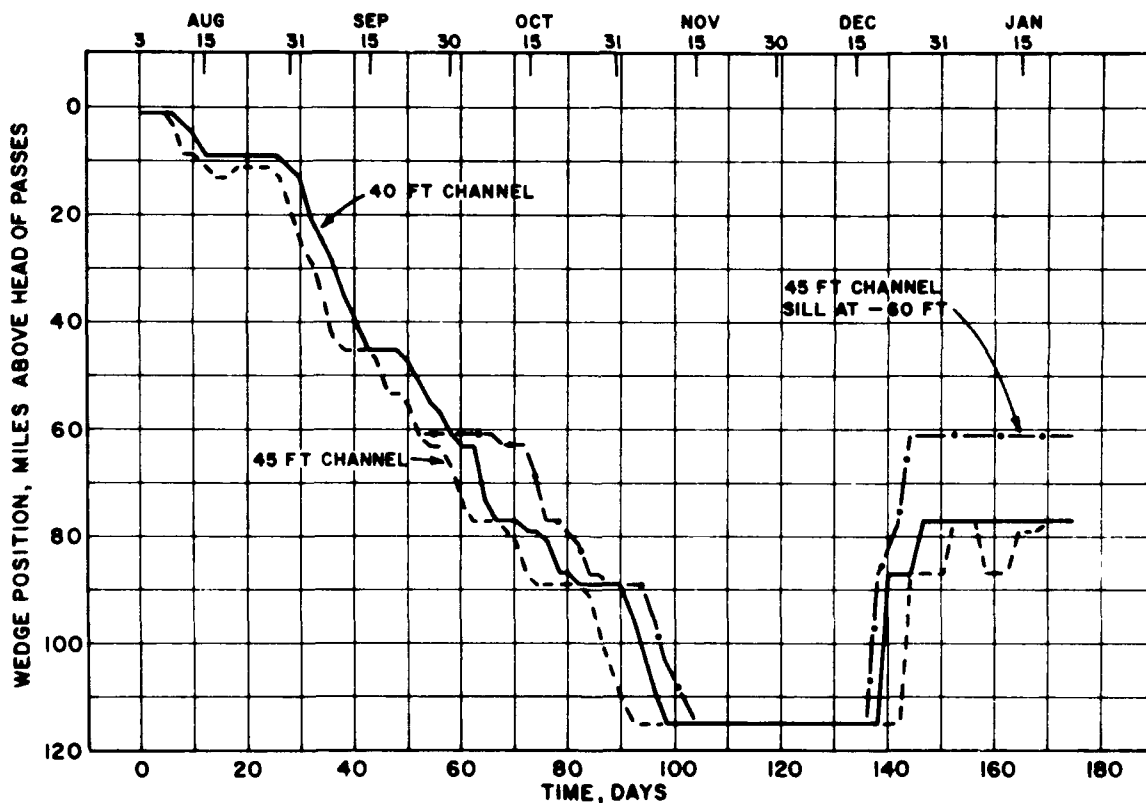


Figure 5. Effect of Sill on Duration of Saltwater Intrusion.

The results from the model were consistent for each historical event modeled and the Corps of Engineers decided to construct a full scale test in the river to convince local governments that the concept was totally feasible and effective. The purposes of the test were threefold: (1) to demonstrate the soundness of the sill concept; (2) to ensure the adequacy of procedures for placing the work under contract in a timely fashion; and (3) to demonstrate that the desired procedures for constructing the sill were valid.

Prior to 1988, river discharges did not decline enough to make a test viable. A key to the plan was to be capable of starting construction at the appropriate time to intercept the saltwater should the river discharge be predicted to decrease to a value that would allow intrusion. The Corps estimated that the sill would require a construction period of four to six weeks and knew it was capable of a reliable river forecast period of only ten days and an outlook of perhaps another 20 days. The toe of the wedge is capable of advancing 2 to 3 miles per day and, for example, in four weeks may travel 60 miles upriver from the Head of Passes. It was crucial that all institutional and logistical requirements be in place before construction could start. In addition, the potential sites must possess sufficient and suitable construction material in the river bed to provide an acceptable fill-to-loss ratio and minimum pumping distance. The most desirable material is medium to coarse sand.

In 1988, the river began falling earlier than at any time before and at such a rate that the wedge had already moved above the first possible site at mile 46 to mile 60 AHP by the 22nd of June. Figure 6 gives a comparison of the 1936 and 1988 discharge hydrographs for the same period.

On 25 June 1988, the decision was made to build the sill at mile 63 AHP. Dredging began on 1 July and by then, the toe of the wedge had moved upstream to mile 80 AHP. Although the salt was beyond the sill site, the sill would temporarially shut off the salt transport as the height of the sill progressed toward the design elevation of 60 feet below the surface. It was at this time that the Corps of Engineers announced the location of the wedge and the purpose for the sill construction. The implications of the announcement were immediate and local and state government officials began to look for ways to replace the municipal water supply of the metropolitan New Orleans area. After devoting local resources to find solutions that were timely, it became evident that there weren't any.
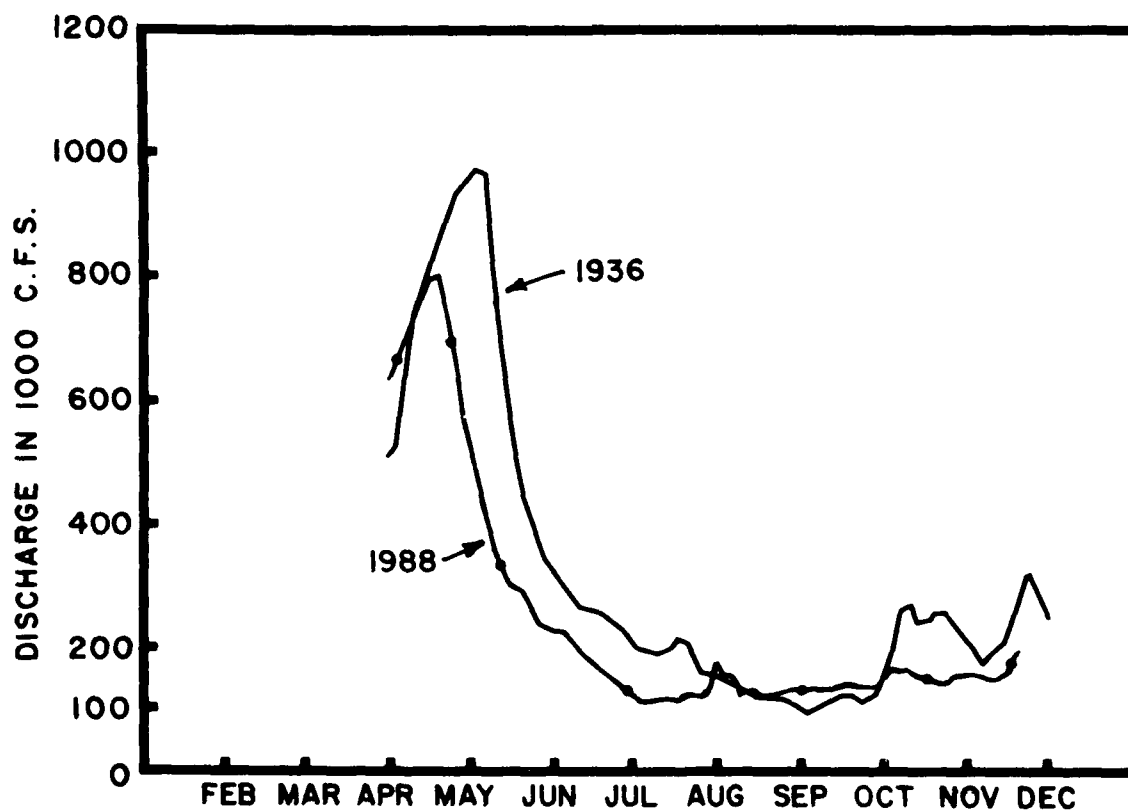


Figure 6. Comparison of 1936 and 1988 Discharge Hydrographs.

The Govenor of Louisiana declared the  area to be in a disaster status on 6 July 1988, and requested Federal assistance through the Office of Emergency Operations of the Corps of Engineers. The Corps agreed to increase the height of the sill to completly shut off the upstream transport of salt provided that this would not impede navigation, the additional construction funds would be authorized, and most importantly, that there would be enough sand available to complete it.  Approval was received on 8 July and orders were issued to the contractor to continue construction to an elevation of -45 feet NGVD (National Geodetic Vertical Datum).  The fill to loss ratio was much higher than originally expected and the sill was  completed to -55 feet NGVD by 10 July and -45 feet NGVD by 22 July.  Full crown width was completed by 1 August.  Table 2 gives pertinent data on important aspects of sill construction. A cross-section of the river and sill are shown in Figure 7.

TABLE 2
Pertinent Sill at Data

| | |
|---|---|
| Design Grade | -45 Ft. NGVD |
| Overall Length @ Crown | 1670 Ft. |
| Average Height | 25 Ft. |
| Maximum Height | 45 Ft. |
| Crown Width | 30 to 115 Ft. |
| Average Side Slopes | 1 Vertical to 7 Horizontal |
| Volume Pumped to Sill | 850,000 Cubic Yards |
| Volume Retained in Sill | 470,000 Cubic Yards |
| Cost | $790,000 |
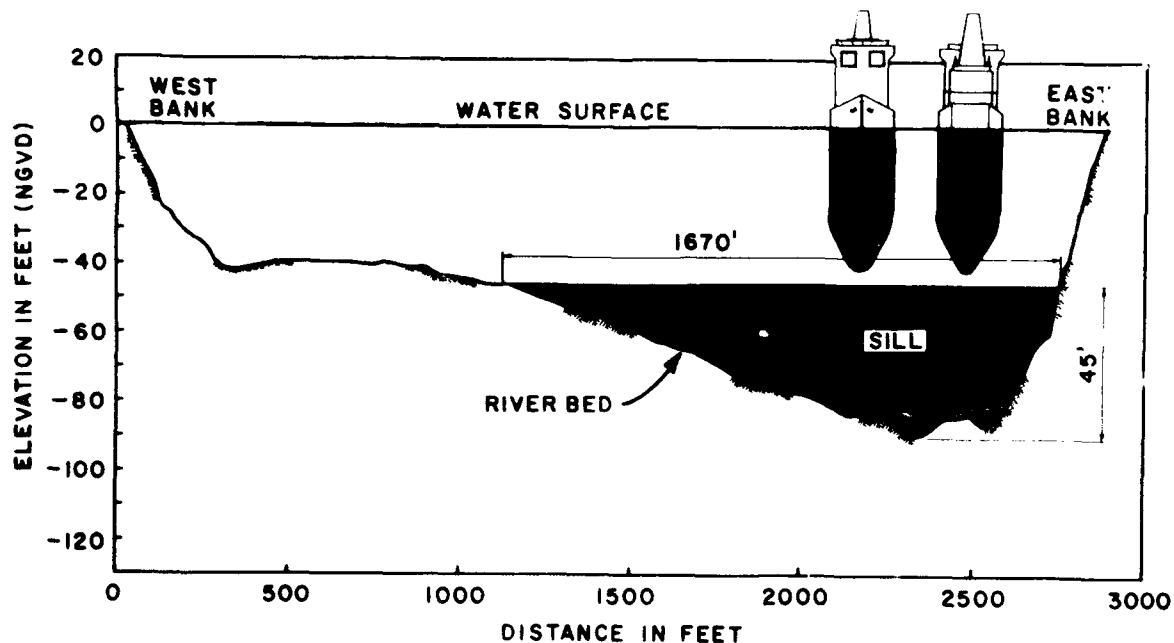


Figure 7.  Typical Cross-section of River and Sill at Mi. 63 AHP.

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## PROJECT MONITORING AND PERFORMANCE

The project monitoring was a joint effort by the Corps of Engineers and the U.S. Geological Survey, at the Federal level, and by the affected water treatment plants at the local level. At the Federal level, hydrologic data were collected four times a week, on Mondays, Tuesdays, Thursdays, and Fridays. This consisted primarily of bottled surface samples and salinity, temperature, and velocity profiles in the vicinity of the wedge toe and immediately upstream and downstream of the sill after construction had been completed. In addition, hydrographic surveys were made almost daily during construction of the sill and biweekly afterwards, and river discharges were measured two to three times per week at an established station some 200 miles upriver from New Orleans. At the local level, the treatment plants took daily bottle samples from the river near the bed, at mid-depth, and at the intake level below the surface, as well as from their finished water reservoirs. These data were then reported to the Corps of Engineers by telephone.

The bottle samples collected by Federal field crews were analyzed on a next-day basis in most cases. Because of the great length of the Mississippi River which had to be monitored, the effort taxed available equipment and manpower in many cases, leaving desirable field data uncollected at points downstream of the sill. As one might imagine, there was great interest from news media, local officials, and industry on a daily basis, and the progression of the salt toe and progress on sill construction were reported as soon as data could be analyzed for accuracy and verified. Figure 8 is a plot of the progression of the toe of the wedge over time. Figure 8 shows that from 11 July to ?7 July, the wedge was arrested at approximately mile 104 AHP, a point adjacent to the major intakes for nearly 1 million people, and intakes located some 15 miles downstream were being impacted as surface water reached a high of 250 ppm Cl⁻ on 11 July 1988. In late July, a much-welcomed rise in river discharge, from a low of 115,000 cfs to 169,000 cfs, occurred as shown on Figure 6, though it was short lived. However, it was of sufficient magnitude and duration to sweep much of the salt downstream and over the top of the sill, and drastically reduced salinity levels at the surface at those intakes located at miles 96, 87, 81 and 76 AHP. Figure 9 shows this for the period 11 July to 12 August in the reach mile 86 to 110 AHP.
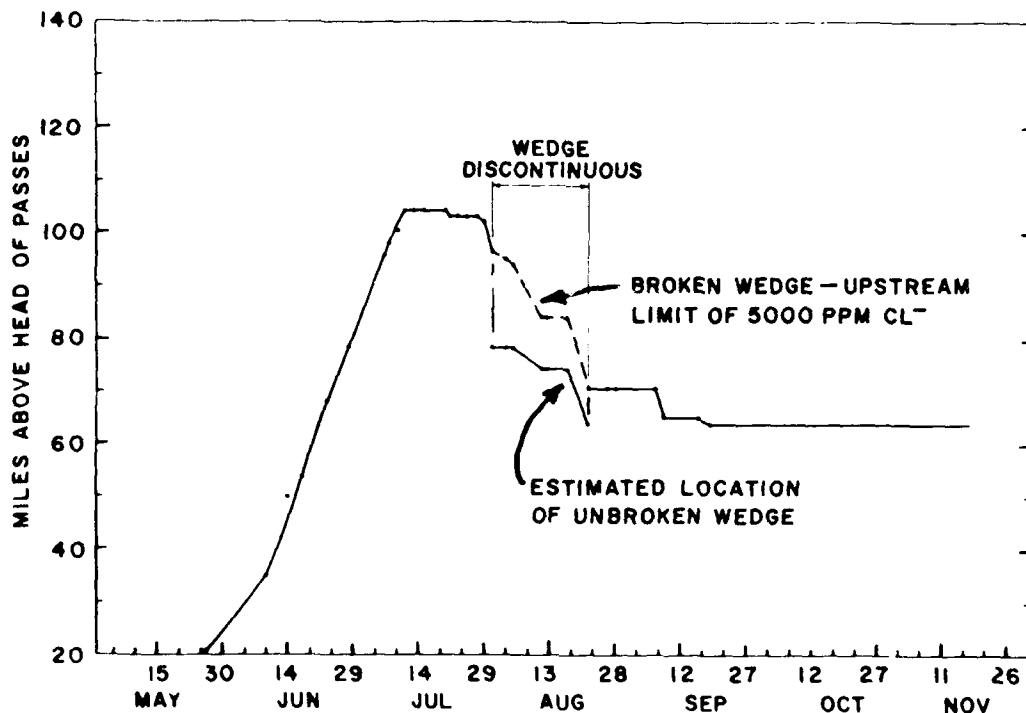


Figure 8. Progression and Regression of the Wedge Toe as a Function of Time.
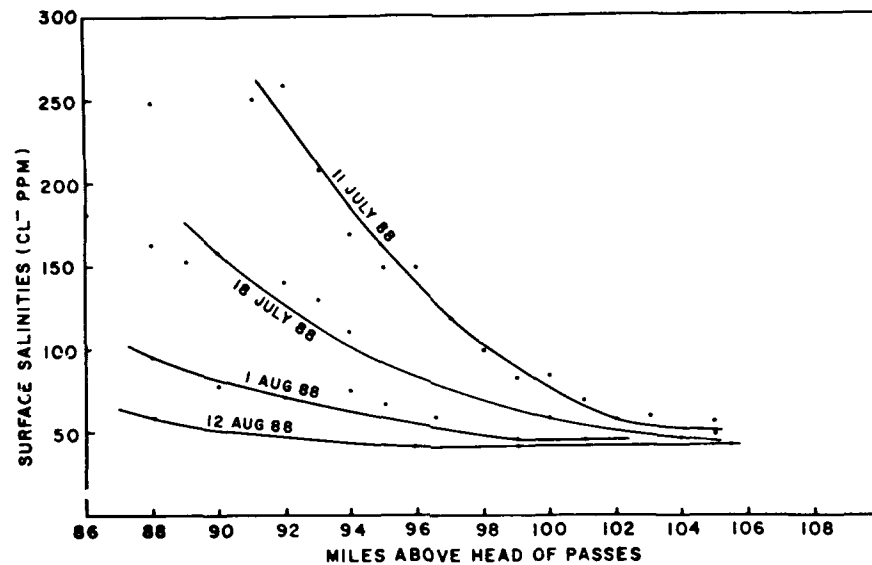
Figure 9. The Reduction of Surface Salinities Over Time.

At this point, the toe of the wedge became difficult to locate due to remnants of trapped saltwater in the deeper holes in the river thalweg which tended to confuse the survey process. Although the sill had effectively shut off any upstream transport of saltwater, the salt in the deep areas of the river persisted until 17 August (see Figure 8). During the period 1-17 August, the wedge was discontinuous and the salt slowly receded to the vicinity of the sill by mid-September where it remained downstream from the sill for the duration.

One of the unknowns which needed to be addressed through prototype construction and monitoring was the resistance of the sand in the sill to erosion which may be caused by ship traffic over the top. Initially, contractual provisions were included which stipulated that the dredge would be subject to call to replace any loss of sill as necessary to maintain the integrity and height of the sill. Semiweekly hydrogrphic surveys demonstrated no tendency for the sill to erode and the dredge was released for other work in October.

Figure 10 shows plots of surveys along the crown of the sill. It can be seen that between 31 July and 14 November 1988, there was no appreciable loss or functional impairment to the sill. It was not until the drought broke near the end of November that high river discharges began to erode the crown. By 5 December 1988, 12 feet had been lost from the top of the sill. On 9 January 1989, the river discharge had reached 668,000 cfs and only one third of the sill was still intact.

The remaining part of the mitigation plan was the barging oi uncontaminated water to those small users who are located downstream of the sill at mile 49 and 18.6 AHP. The Corps of Engineers and the local government, beginning approximately 11 July, used eleven 35 by 192-foot steel barges to transport 131 million gallons of fresh water to these users for blending or direct treatment over a period of 122 days. The average cost delivered was less than 2 cents per gallon and fully met community needs.

SUMMARY

Subnormal flow in the Mississippi River persisted from the end of April to 25 November 1988. Despite a late start to block the saltwater, the sill was completely effective and protected, in aggregate, approximately 26 billion gallons of water at a cost of $790,000. This yielded a cost per gallon of three thousandths of a cent. Alternate solutions would have cost more than $50 million by comparison. Table 3 gives a comparison of surface salinities with and without the sill at key freshwater intakes in metropolitan New Orleans. Columns (4) and (5) give surface salinities in ppm Cl.

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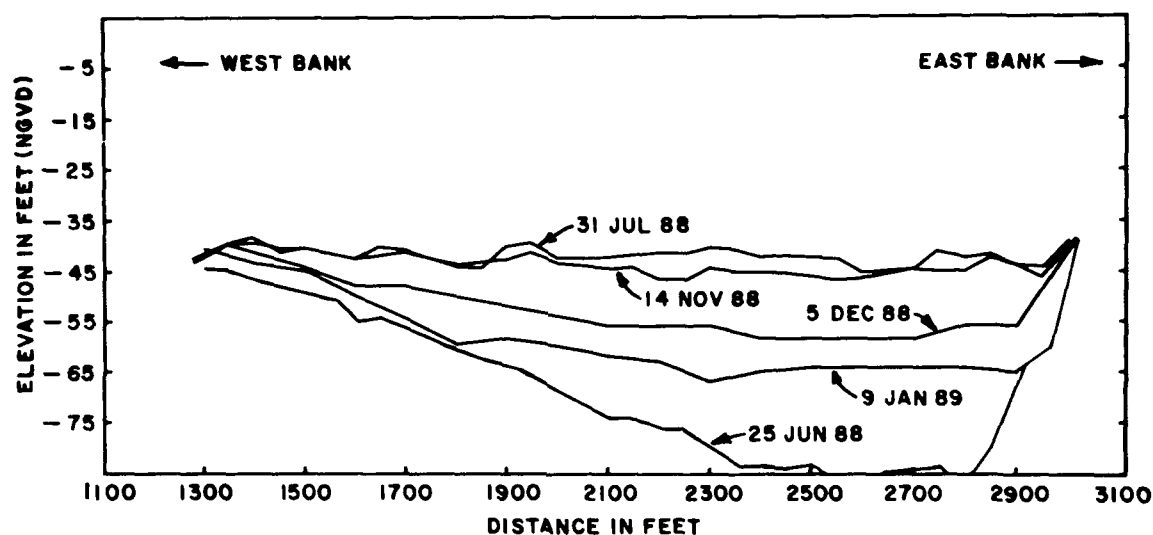
Figure 10. Resistance of Sill to Erosion Over Time Due to Flow

TABLE 3
Surface Salinities for With and Without the Sill

| Location<br>(1) | Miles AHP<br>(2) | Date,1988<br>(3) | Chloride,ppm<br>With Sill<br>(4) | Chloride,ppm<br>No Sill<br>(5) |
|---|---|---|---|---|
| Dalour | 81 | 15 Jul | 385 | 385 |
| | | 15 Aug | 61 | 600 |
| | | 15 Sep | 40 | 750 |
| | | 15 Oct | 54 | 900 |
| | | 15 Nov | 72 | 750 |
| | | 1 Dec | 45 | 250 |
| St.Bernard | 88 | 13 Jul | 255 | 255 |
| | | 15 Aug | 53 | 500 |
| | | 15 Sep | 53 | 625 |
| | | 12 Oct | 60 | 800 |
| | | 15 Nov | 66 | 625 |
| | | 29 Nov | 48 | 250 |
| New Orleans<br>and<br>Jefferson | 105 | 12 Jul | 46 | 46 |
| | | 15 Aug | 42 | 285 |
| | | 15 Sep | 41 | 380 |
| | | 12 Oct | 47 | 500 |
| | | 23 Nov | 42 | 250 |

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## CONCLUSIONS

The results achieved from construction of the sill were generally similar to those predicted by the mathematical model, LAEM, but the duration of the drought subjected the project to a more rigorous hydrologic event than the records would indicte possible and laid aside any doubts that the concept might be impractical or uneconomical. This field experience demonstrated that the sill: (1) can be constructed efficiently under a wide range of field conditions; (2) will remain stable and functional over a long period of sustained drought; and (3) will erode, as had been predicted by the modeling effort, only when the river discharge increases to 400,000 cfs. The concept can be applied to problems of saltwater intrusion in well-stratified systems elsewhere with a high degree of confidence, provided that boundary conditions are reasonably similar.

# A 3-D NUMERICAL WATER QUALITY MODEL EVALUATION
# OF LOS ANGELES AND LONG BEACH HARBOR ENHANCEMENTS

by

Ross Hall[1]

## Introduction

Los Angeles and Long Beach Harbors (San Pedro Bay) are located adjacent to each other and share a common breakwater system (Figure 1). The Ports of Los Angeles and Long Beach have undertaken a long-range cooperative planning effort known as the 2020 Plan. Incorporated in the plans are harbor enhancements that include dredging and new landfill.

Harbor enhancements may affect water quality in the study area by changing tidal circulation and flushing patterns. The major water quality concern is the dissolved oxygen (DO) resource. Channel deepening introduces the possibility of DO stratification where well-oxygenated surface waters overlie oxygen-depressed bottom waters.

The purpose of this study was to compare the flushing and DO resources of existing and four plan conditions through numerical model simulations. A three-dimensional water quality model (WQM) was used to address possible vertical DO stratification. Hydrodynamic output from the 3-D hydrodynamic model CH3D was used as input by the WQM to transport water quality constituents.

## Water Quality Model

The WQM selected for this study was a modification of the Environmental Protection Agency WASP code (Ambrose, Vandergrift, and Wool 1986). Major adaptations included (a) improved adjective and diffusive transport schemes, (b) provisions for the input and processing of CH3D three-dimensional hydrodynamic data, and (c) implementation of kinetic routines specific to the San Pedro Bay application.

### Adjective and Diffusive Transport Schemes

Horizontal flows were distinguished from vertical flows, and improved transport schemes were implemented. The horizontal advective transport scheme used was a modified version of the third order QUICKEST scheme (Leonard 1979, Hall and Chapman 1985, and Chapman 1988). QUICKEST eliminates the problems of excessive numerical diffusion characteristic of the upwind difference scheme originally used in the WASP code.

For vertical transport, an implicit vertical adjective and diffusive transport scheme was implemented. Central differences were used for both vertical advection and diffusion terms. This scheme removed the stability restriction associated with small vertical cell lengths.

### Model Linkage

The hydrodynamic and water quality models were linked by spatially and temporally averaging CH3D output to drive the WQM. The hydrodynamic model used extensive spatial resolution to resolve geometric features of the harbors. The CH3D spatial resolution was of the order of 100 meters and required a time step of 60 seconds for stability. In contrast, the WQM has characteristic time scales

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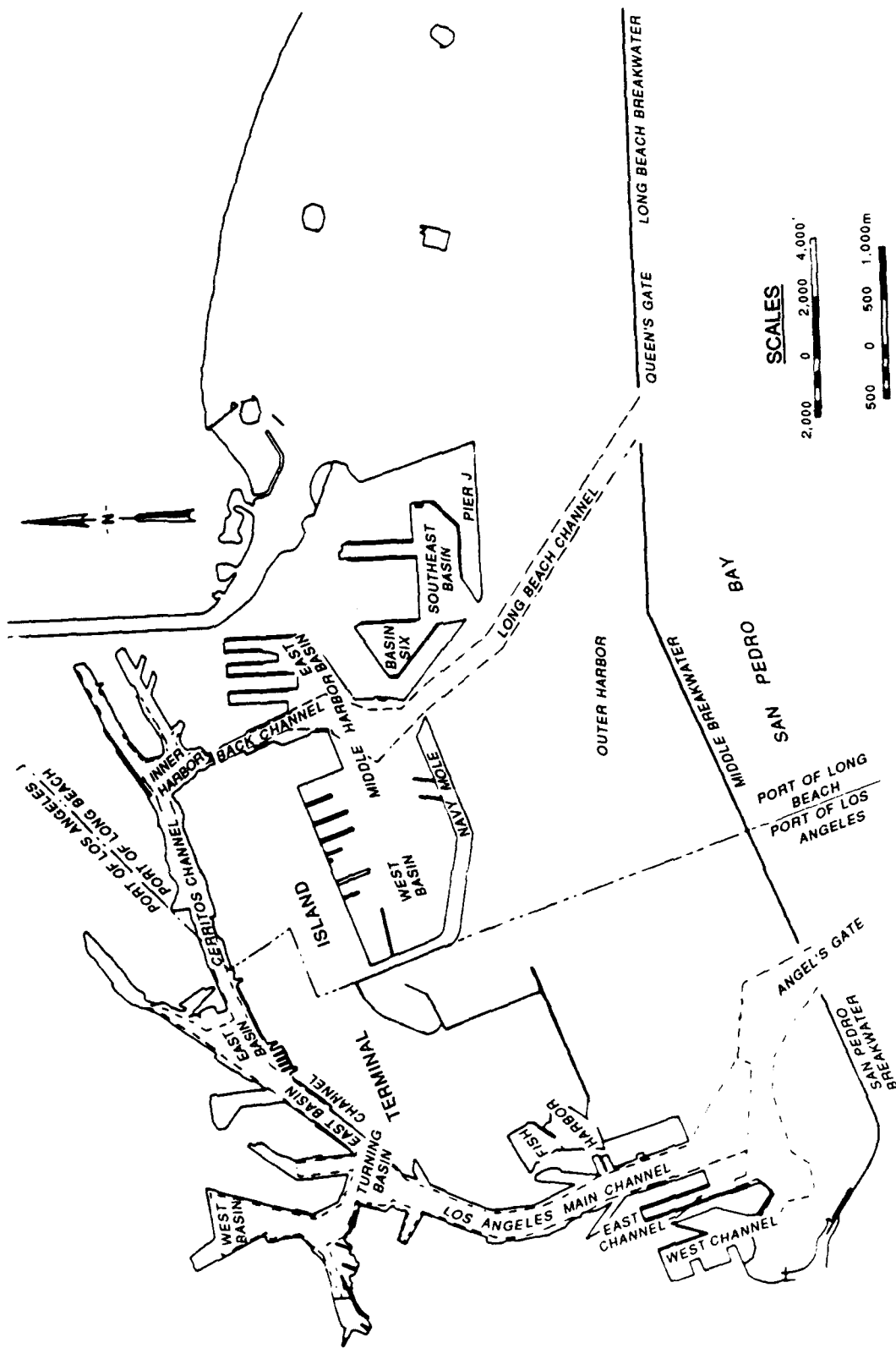
Figure 1. Location of channels and basins in Los Angeles and Long Beach Harbors, California

determined by the kinetic rate coefficients on the order of hours. Water quality analyses allow a spatial resolution an order of magnitude greater than that used by CH3D.

Since the WQM required less spatial resolution than CH3D, spatial characteristics of the hydrodynamic grid such as volume, cell surface areas, cell facial areas, and cell lengths were summed, resulting in an "overlaid" WQM grid. Figure 2 is a schematic of the WQM grid overlying the CH3D grid for the existing conditions. The WQM grid maintained the same vertical resolution as the CH3D grid of three vertical layers.

Overlaying the WQM grid decreased the number of computational cells by a factor of 16. The larger WQM cell sizes allowed an increase of the time step used from 60 to 900 sec while maintaining computational stability. Reductions in spatial resolution greatly reduced the computer time required for the WQM.

Tracer tests were used to guarantee that the WQM was preserving transport properties of CH3D. Tracer injections were made to several hydrodynamic grid cells in the surface layer. These cells corresponded to a single WQM cell. The behavior of the tracer was simulated over several days in both models and the decline in tracer concentration was in excellent agreement (Hall 1989).

Subtidal oscillations, characterized by a pulsating flow pattern with areas of flow direction reversal, were simulated in CH3D. The flow pulsations occurred at a frequency of 1 hour. A concern that capturing the subtidal oscillations may be important in the WQM was investigated by another tracer test (Hall 1989). Tracer injections were simulated with both a 15- and 60-minute average of CH3D results. Decline of tracer concentrations was the same in both simulations which confirmed that a 60-minute average is sufficiently accurate for WQM use.

Kinetic Routines

The water quality model study focused on dissolved oxygen resources and flushing characteristics of the harbor system. The WQM simulated the following variables: dissolved oxygen (DO), carbonaceous biochemical oxygen demand (CBOD), ammonia nitrogen ($NH_4$-N), nitrite plus nitrate nitrogen ($NO_2$ + $NO_3$-N), algal biomass as carbon, orthophosphate ($PO_4$-P), and a conservative tracer. Temperature and salinity were held constant.

The water quality kinetic algorithms were adapted from the WES two-dimensional, laterally-averaged model of hydrodynamics and water quality, CE-QUAL-W2 (Environmental and Hydraulics Laboratories 1986), and HydroQual's Potomac Eutrophication Model (Thoman and Fitzpatrick 1982). The water quality kinetic routines are detailed in Hall (1989).

Flushing Studies

Flushing studies were used to provide a qualitative comparison between existing and the plan enhancement schemes. A decrease in the flushing rate prolongs the period of time that oxygen-demanding substances exert their influence on the DO concentration. A decrease in flushing rate can intensify other potential water quality problems and indicates that more detailed water quality analyses are required.

The initial test involved injecting a tracer throughout the harbor, inside the outer breakwater, and simulating its behavior over a 25-day period. Any tracer leaving the harbor is assumed lost so the decline of tracer concentrations gave an indication of the flushing characteristics of the harbor. This test indicated three areas of potential problems: East Basin, embayment between West Basin and Fish Harbor, and Navy Harbor.

Separate tests were then made by injecting tracer into a confined area of each potential problem basin. These tracer tests revealed that East Basin flushes more rapidly in the enhancement schemes

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# SAN PEDRO BAY - EXISTING

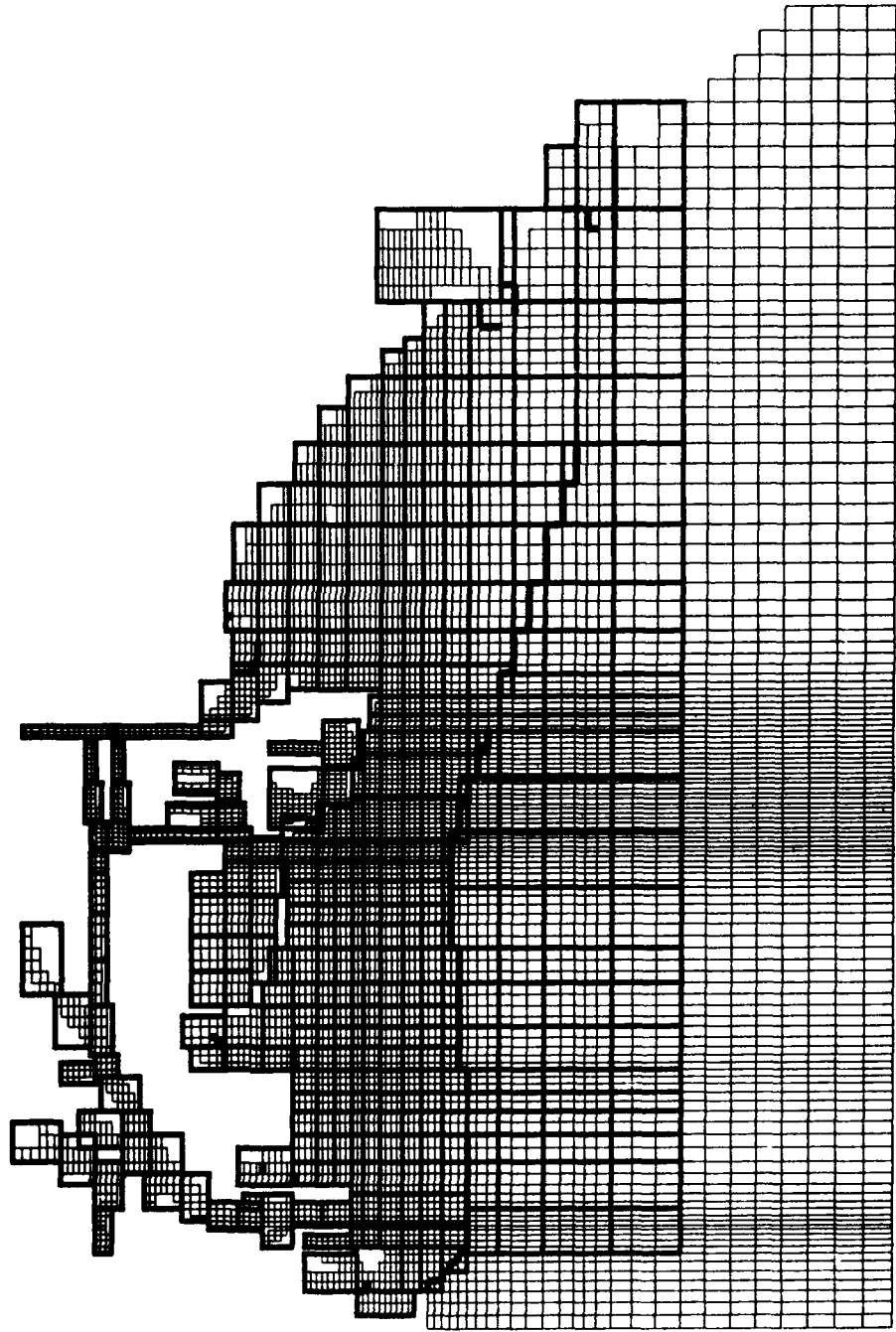## Water Quality Model Grid Overlaid



Figure 2. Overlay of water quality grid on hydrodynamic grid

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than in the existing condition and that the embayment between West Basin and Fish Harbor and Navy Harbor flush less rapidly. Minor differences were noted between enhancement schemes. The tracer studies indicated that circulation through Los Angeles Main Channel is rather static, but slightly counterclockwise in the existing condition and clockwise in the plan conditions. These tests also provided information on where to examine impacts in the water quality simulations.

## Water Quality Studies

Kinetic constants, boundary conditions, and initial conditions used are detailed in Hall (1989). Boundary conditions included observed water quality at the ocean boundary*, measured sediment oxygen demand (SOD)*, light exchange and reaeration through the surface, and water quality of the Terminal Island Treatment Plant (TITP) discharge. The TITP discharge was simulated for the existing condition, but was not in the plan simulations. Initial conditions were specified by assuming horizontally constant yet vertically stratified water quality based on the water quality sampling program conducted during August 1987 by Tekmarine, Inc.*

The initial values represented averages measured during the first week of August 1987. No algae or CBOD was detected during the first week of sampling. Therefore, the monthly average data values of algae were used for initial conditions. The observed data show the harbor system is nitrogen-limited and with little water column respiration. Therefore, the most important concerns were benthic respiration or sediment oxygen demand and water residence time or flushing characteristics. CH3D was used to simulate hydrodynamics for the entire month of August and time-averaged results were used by the WQM.

Figure 3 displays simulated algae (Alg), orthophosphate ($PO_4$-P), ammonia nitrogen ($NH_4$-N), nitrite + nitrate nitrogen ($NO_2 + NO_3$), 5-day carbonaceous biochemical oxygen demand (CBOD5), and dissolved oxygen (DO) at the bottom at Station I-2 which was located in Outer Harbor near Fish Harbor. The solid line represents the existing condition (Base) and the interrupted lines represent the different enhancement conditions (Scheme A through Scheme D). The circles represent the observed values. The simulated period extended from August 1, 1987 (Julian Day 213) through August 28, 1987 (Julian Day 240). The measured and simulated results are in general agreement.

Simulated DO of the enhancement plans was either equal or slightly less than existing conditions. Maximum deviations of 0.5 g/m**3 occurred in the Long Beach Channel. It should be noted that the tracer simulations revealed decreased flushing in this area.

The bottom waters exhibited lower DO relative to the surface waters. The maximum deviations between surface and bottom waters occurred in Cerritos Channel, Back Channel, Middle Harbor, East Basin of Middle Harbor, and the dead-end channels connected to Inner Harbor. Only the stations in Back Channel and Middle Harbor exhibited differences between existing and enhancement conditions. Differences in DO between existing conditions and enhancement plans were not greater than 0.5 g/m**3. The minimum predicted DO at all stations and all depths was 6.0 g/m**3. The minimum of 6.0 g/m**3 was predicted in the bottom layer in West Basin of Middle Harbor.

The only differences in DO between enhancement plans occurred near the junction of Outer Harbor and the Los Angeles Main Channel. One plan displayed surface DO about 0.25 g/m**3 greater and bottom DO about 0.5 g/m**3 greater than the other plans. Study results are presented in Vemulakonda, Hall, and Chou (1989).

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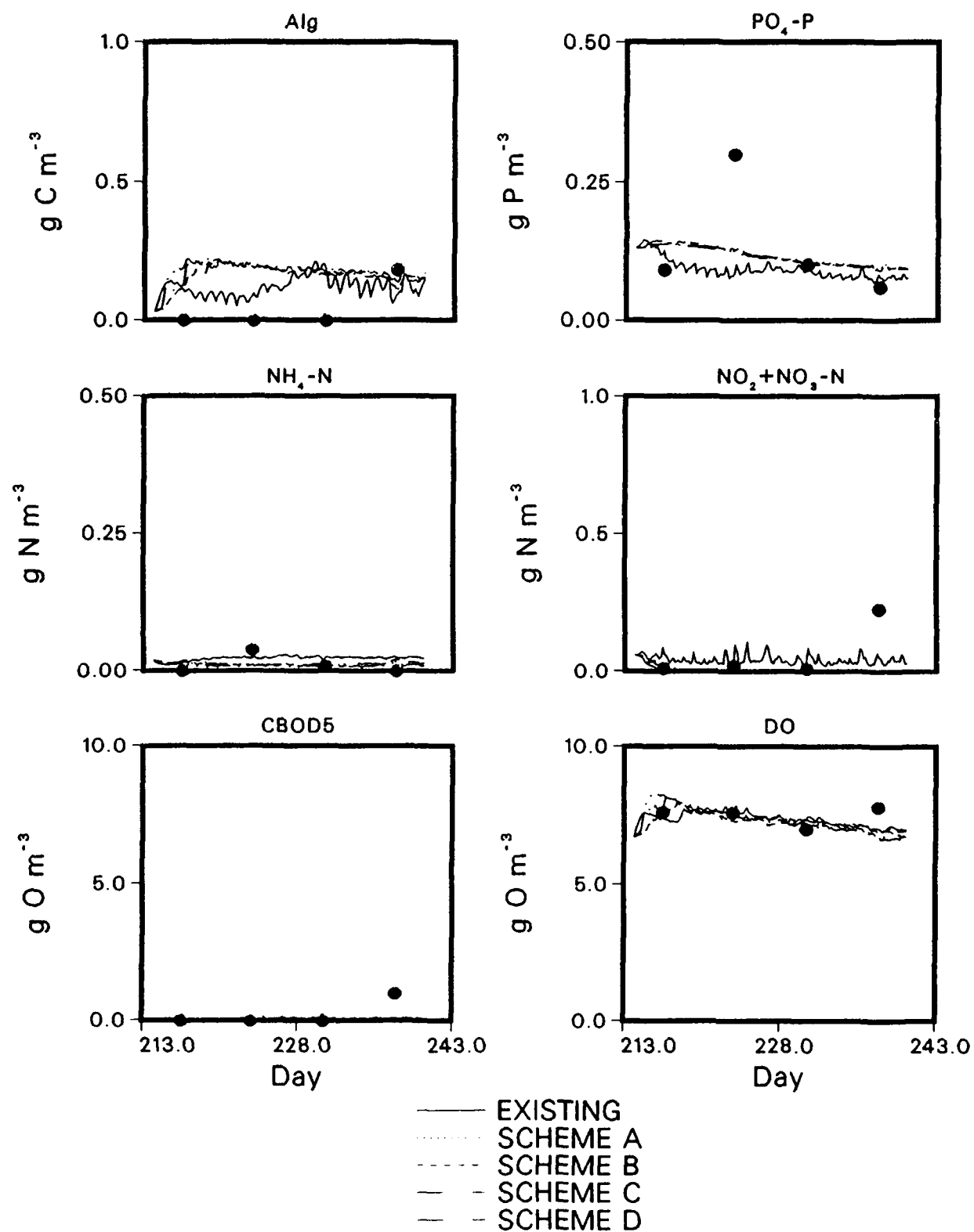
# WQ Station I- 2
## Bottom

Figure 3.   Existing and plan simulations of water quality

## Summary and Conclusions

The results of this model study indicate that the plans for Los Angeles and Long Beach Ports harbor enhancement will reduce circulation and flushing in several areas, such as Navy Harbor and the embayment between West Basin and Fish Harbor. Residual circulation in the Main Channel is expected to change from counterclockwise (existing conditions) to clockwise for the enhancement schemes. The enhancement schemes will result in less rapid flushing in the Inner Harbor-Back Channel-Middle Harbor of the Port of Long Beach.

The main DO impacts of the enhancement plans are experienced in the Inner Harbor-Back Channel-Middle Harbor of the Port of Long Beach. Simulated differences did not exceed 0.5 g/m**3. The bottom waters exhibited lower DO than the surface waters, but simulated concentrations were greater than 6.0 g/m**3. The simulated DO indicated little difference between enhancement plans. The only difference observed was that one plan provided slightly greater DO at the junction of Outer Harbor and the Los Angeles Main Channel (0.25 g/m**3) and in the West Channel of Los Angeles Harbor (0.5 g/m**3). However, because of the observed small deviations, no plans are preferred for DO.

## Acknowledgements

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# CONFINED DISPOSAL AREA EFFLUENT QUALITY
# FOR NEW BEDFORD SUPERFUND PILOT DREDGING

by

Michael R. Palermo[1] and Edward L. Thackston[2]

## BACKGROUND

### Modified Elutriate Test

Confined disposal facilities (CDFs) are commonly used for disposal of dredged material containing a wide range of contaminants. The effluent discharged from these areas during hydraulic filling operations is a potential pathway for contaminant release. Testing requirements for Section 404 of the Clean Water Act call for use of a modified elutriate test for predicting the water quality of effluent. The modified elutriate test procedure was developed as a part of the Long Term Effects of Dredging Operations (LEDO) program (Palermo 1985, 1986).

The test procedure determines both the dissolved and total concentrations of contaminants in the elutriate. A separate column settling test, developed to evaluate the settling design for CDFs, is used to determine the total suspended solids (TSS) concentration of the effluent for a given set of operational conditions. Results from both the modified elutriate and settling column tests can then be used to predict the total concentration of contaminants in the effluent. A schematic diagram of the modified elutriate procedure is shown in Figure 1.

### Prior Field Verification

Data from five field evaluations on maintenance dredging projects confirmed that the test is a reliable and conservative predictor of effluent quality (Palermo 1988, Palermo and Thackston 1988a, 1988b). At the five sites used for verification (Mobile Harbor, Savannah Harbor, Norfolk Harbor, Black Rock Harbor in Bridgeport, Connecticut, and Hart-Miller Island near Baltimore), the primary pollutants found in the sediments were heavy metals and nutrients. The means of the ratios of laboratory-predicted to field-measured concentrations of heavy metals and nutrients were 2.5, 1.7, 1.7, 1.4, and 1.6 for the five sites, verifying that the modified elutriate test and predictive technique accurately predicted effluent water quality.

However, only one sediment exhibited a high enough concentration of a specific organic pollutant to be found in the effluent. The total PCB concentration in the effluent from the Black Rock Harbor site was 0.0099 mg/l, versus a predicted value of 0.013 mg/l.

Since 1985, a site has been sought for additional field evaluation involving a sediment contaminated with one or more specific organic compounds at high enough concentrations that a valid comparison of the predictive ability of the test and field data for organics could be conducted. A Superfund project at New Bedford, Massachusetts, where the sediments in the harbor are highly polluted with PCB, presented such an opportunity.

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[2]Professor and Chairman, Department of Civil and Environmental Engineering, Vanderbilt University.
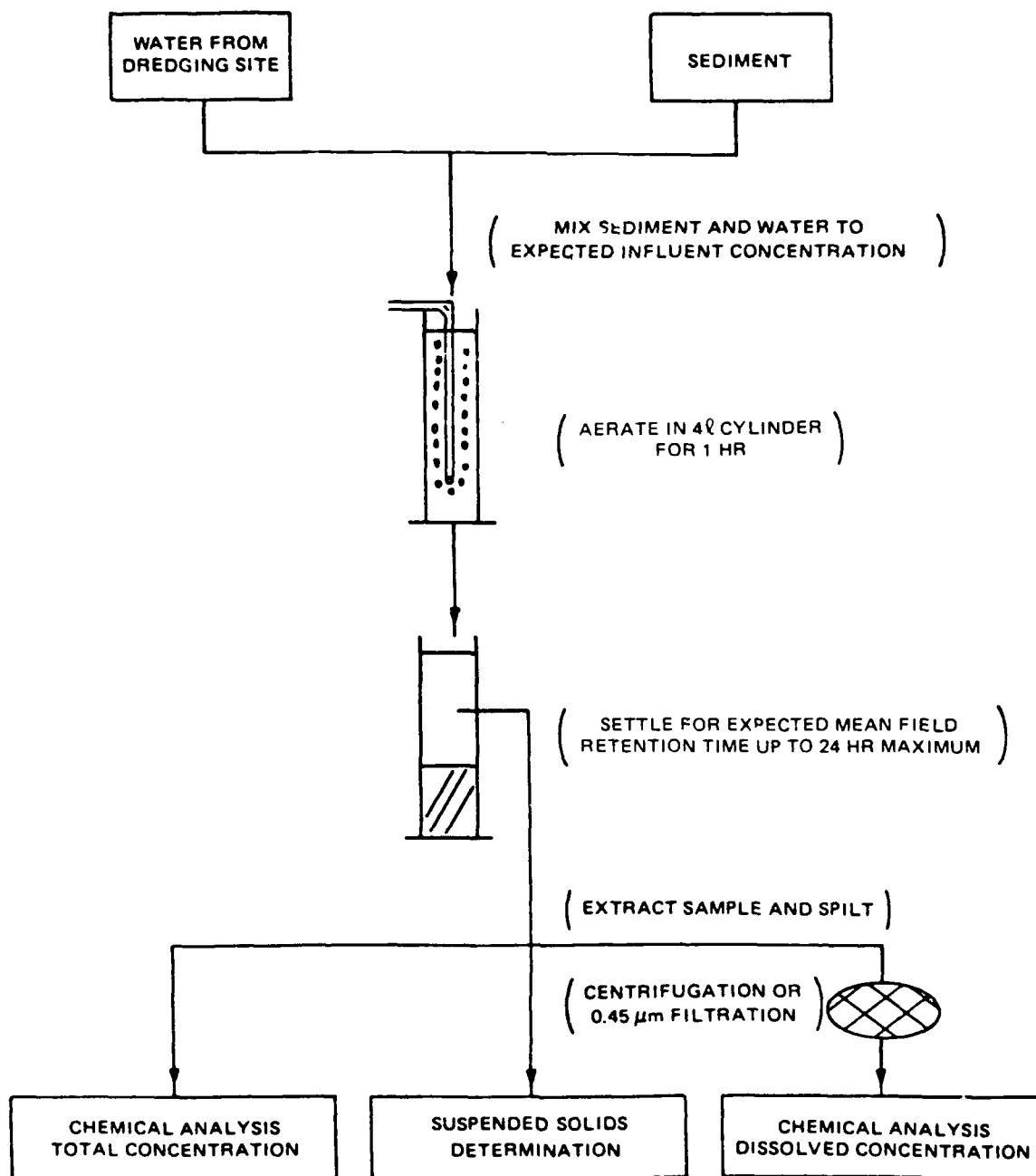
Figure 1. Schematic diagram of the modified elutriate test.

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# NEW BEDFORD SUPERFUND EFFLUENT EVALUATION

## Project Description

The harbor of New Bedford, Massachusetts is highly contaminated by PCBs discharged by industries in the years prior to the 1980's. PCB concentrations in the sediments range from a few parts per million (ppm) to over 100,000 ppm. The harbor was designated a Superfund site in 1982.

In August 1984, the Environmental Protection Agency (EPA) published a Feasibility Study of Remedial Action Alternatives. The study proposed five alternatives for cleanup, four of which involved dredging of the estuary to remove the contaminated bottom sediments. An Engineering Feasibility Study (EFS) was performed by the Waterways Experiment Station (WES) and the New England Division (NED) (Francingues and Averett 1988). Conceptual designs of dredging and disposal alternatives were developed and evaluated for their implementability and potential for contaminant release (Averett, et al. 1988).

The alternatives were field tested during a pilot field-scale test from May 1988, to February 1989 (U.S. Army Engineer Division, New England 1989). One of the five alternatives investigated involved dredging 2,200 yd3 of contaminated material from the estuary, followed by 3,900 yd3 of cleaner cap material, and placing it in a CDF on shore. The CDF, shown in plan view in Figure 2, was a diked area acting as a sedimentation basin. It consisted of two cells, with the discharge of clarified water back into the harbor. The effluent concentration from the primary cell is the appropriate field concentration for comparison with the modified elutriate data.
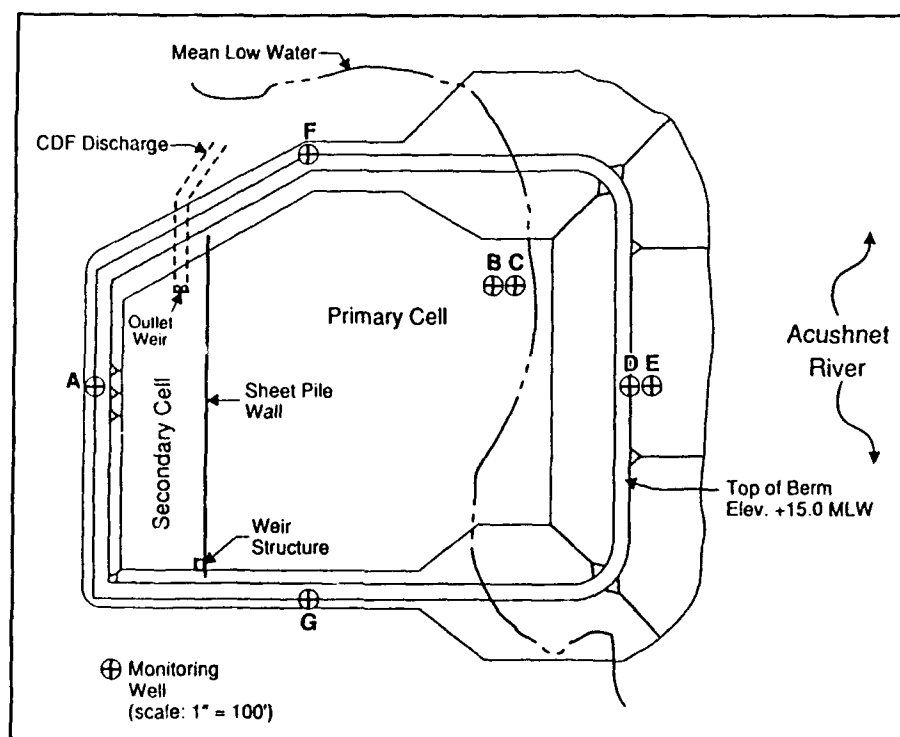


Figure 2. Plan of Confined Disposal Facility for New Bedford Superfund Pilot Study.

Effluent PCB Predictions

Two replications of the modified elutriate test were performed by WES on a sample of sediment from the area to be dredged. PCB analyses of the elutriate samples were performed by NED. Total PCB, dissolved PCB, and TSS were measured in each replicate. A third replicate was invalidated because of a laboratory decantation error. A laboratory column settling test was also performed on the sediment sample from the area to be dredged. The predicted TSS concentration for the effluent from the primary cell, for the ponding conditions and detention time during the pilot study, was 120 mg/l. A dissolved concentration of 7.5 ppb PCB in the effluent was predicted, based on the modified elutriate test, while the fraction of PCB in the TSS was predicted as 482 mg/kg TSS (dry weight basis). A total concentration of 65 ppb PCB in the effluent was predicted, based on the results of both the modified elutriate test and column settling test.

Field Sampling

During the time that contaminated material was being pumped into the CDF during the field test, the primary weir was sampled hourly for 12 days. All samples were analyzed for TSS, and daily composites were analyzed for TSS, PCB, and metals (Cu, Cd, Pb). This sampling and testing effort was performed by the EPA Environmental Research Laboratory (ERLN) at Narragansett, R.I.

Daily composite TSS at the primary weir ranged from 35 to 136 mg/l, and averaged 67 mg/l. Dissolved PCB concentrations in the primary cell effluent ranged from 0.6 to 4.3 ppb and averaged 1.9 ppb. Total PCB concentrations in the primary cell effluent ranged from 6.5 to 17.2 ppb and averaged 11 ppb as measured by ERLN. Calculated values of the fraction of PCB in the effluent TSS ranged from 60 to 454 mg/kg TSS and averaged 170 mg/kg TSS.

Comparison of Laboratory-Predicted and Field-Measured Data

The laboratory-predicted value of dissolved effluent PCB is higher than the field-measured value by a factor of 3.9 (7.5 ppb versus 1.9 ppb), indicating that the modified elutriate test was a conservative predictor of dissolved effluent PCB for this site. The laboratory-predicted value of the fraction of PCB in the effluent was higher by a factor of 2.8 as compared with the field-measured value (482 versus 170 mg/kg TSS), indicating that the modified elutriate test was also conservative in predicting the particle-associated concentrations of PCB. The prediction of effluent TSS, based on column settling tests, was higher by a factor of 1.8 (120 mg/l versus 67 mg/l), showing that the laboratory column sedimentation test was also a conservative predictor for the operating conditions and sediment at this site. The resulting predicted total concentration of PCB in the effluent was conservative by a factor of 5.9 (65 ppb versus 11 ppb), reflecting the degree of conservatism in both the modified elutriate and column settling tests.

A different comparison of the laboratory-predicted and field-measured data can be made considering the fact that different instrumental techniques were used in the chemical analyses. NED, which conducted the modified elutriate test, used a gas chromatograph with a packed column. ERLN used a long capillary tube in lieu of the packed column. These two techniques yield differing resolution of the chromatograph, and, based on differing calibrations, may yield differing results. For the New Bedford pilot study, these two techniques yielded consistently different results, as evidenced by twenty-two split samples taken from the silt plume around the dredging operation. These split samples were analyzed by both labs. The relationship between the PCB concentrations for the split samples is shown in Figure 3. Unfortunately, no split samples of the effluent itself were analyzed. Without commenting on the question of which instrumental technique is best, or more accurate, it can be stated that the two labs consistently obtained quite different results.

The ERLN-measured value of 1.9 ppb PCB for dissolved effluent samples has an adjusted value of approximately 4.8 ppb, using the relationship in Figure 3, indicating that the modified elutriate test overpredicted the dissolved concentration by a factor of 1.6 (7.5 ppb versus 4.8 ppb). The computed fraction of PCB in the TSS, based on the ERLN field samples has an adjusted value of 425 mg/kg TSS,

very close to the computed value for modified elutriate tests (482 versus 425 mg/kg), indicating that the modified elutriate test was an accurate predictor of the particle-associated concentrations of PCB. The ERLN-measured value for total concentration of PCB in the effluent has an adjusted value of 28 ppb, using the relationship in Figure 3, indicating the predicted value was conservative by a factor of 2.8 (65 ppb versus 28 ppb), reflecting the degree of conservatism in the column settling tests.
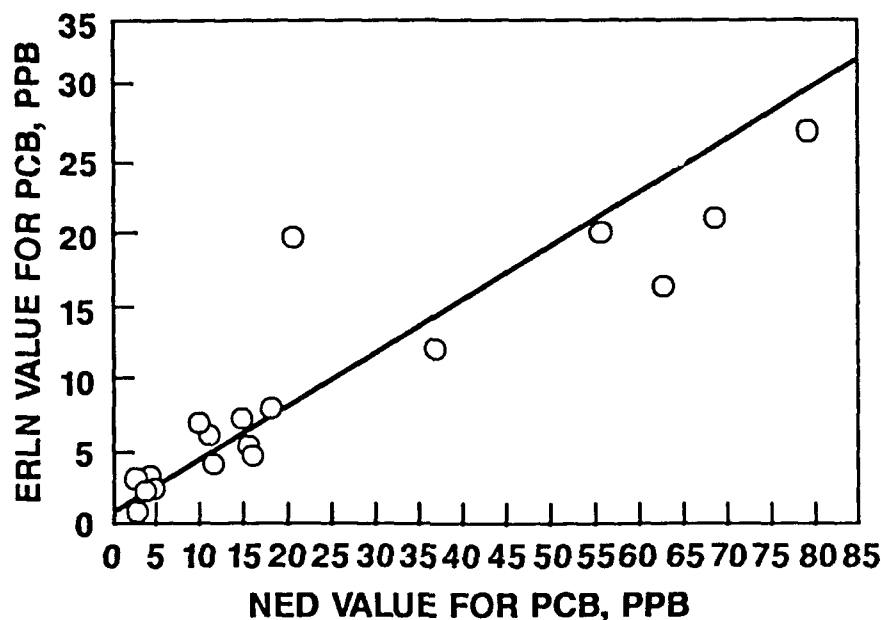


Figure 3. Relationship between PCB concentrations measured by ERLN and NED.

## CONCLUSIONS

The comparison of laboratory-predicted and field-measured concentrations of PCB can be interpreted in two ways because of the fact that the lab analyzing the field samples used a different instrumental technique for PCB analyses than the lab which ran the predictive test. Based on either of the interpretations, the modified elutriate test produced useful and accurate predictions of the concentration of PCB in the effluent from a CDF receiving dredged sediments highly contaminated by PCB. The predictive technique, using both the modified elutriate and column settling test, was a conservative predictor of effluent water quality for PCB for this site.

## ACKNOWLEDGEMENT

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# PCB VOLATILIZATION FROM DREDGED MATERIALS
## INDIANA HARBOR, INDIANA

by

Jay Semmler[1]

## INTRODUCTION

Volatilization is the process whereby a compound passes into the air from a solid or liquid surface. The degree of volatilization can be generally related to the vapor pressure of the compound: a compound with a high vapor pressure would volatilize rapidly, while one with a low relative vapor pressure would volatilize slower.

To date, no simulative models have been developed to predict volatilization from a sediment/soil type material. The model presented in this paper provides an estimate of the mass of PCBs lost from an inlake and an upland Confined Disposal Facility (CDF). PCB was the only compound considered due to its regulatory significance and to simplify development of the model. It is anticipated that other semivolatile and volatile compounds such as polycyclic aromatic hydrocarbons (PAHs) will be modeled in the future for sediments contaminated with these substances.

Typically, chemical equilibrium principles are used to determine the transfer of the volatile organic chemicals (VOCs) between various phases. In the case of VOCs associated with sediment, three phases of matter are involved. These are the solid particles which constitute the sediment and include both organic matter and mineral matter comprising the particles. The two other primary phases include air and water (USACE, 1988).

With respect to dredging, VOCs can enter the air from either the water or sediment surfaces. For volatilization to occur from the water surface, the VOC must first desorb from the suspended solids phase and diffuse through the water before being emitted into the air.

## MODEL PURPOSE

A PCB volatilization model has been developed by the Chicago District as part of the preparation of the Draft Environmental Impact Statement (DEIS) for Indiana Harbor and Canal Maintenance Dredging and Disposal Activities Lake County, Indiana. The model predicts the mass flux of PCBs through volatilization from dredged materials disposed in a proposed CDF. Two scenarios were considered: the first assumes that the dredged materials are placed in an inlake CDF, while the second assumes placement in an upland CDF.

Volatilization can involve a complicated interconnected number of transfer pathways. In order to quantify volatilization, one needs to address all sources, pathways, and external parameters which effect the transfer. At this time, lab and field verification of critical transfer coefficients is lacking, and therefore a complete quantification of PCB volatilization for all the activities associated with a dredging operation is beyond the scope of this report. In light of this, the model is used as an indication of the relative significance of volatilization when compared to other loss pathways (leachate, seepage, plant and animal uptake etc...) for various operational schemes. In this manner a ranking of potential PCB mass flux for different disposal options can be determined and viable options can be evaluated against each other and the no-action plan.

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## MODEL ASSUMPTIONS

This is the first known attempt to simulate the emission of PCBs into the air from a CDF. Theoretical chemodynamic models for organic pollutants in dredged material are used to estimate potential emission rates of PCBs to the air. Although the chemodynamic models have not been verified experimentally for dredged material, studies of pesticide volatilization from soils, VOC emissions during refinery waste landfarming, and VOC emissions from hazardous waste lagoons indicate that theoretical chemodynamic models, when properly formulated, provide realistic estimates of VOC volatilization (Thibodeaux and Hwang, 1982; Thibodeaux and Becker, 1982; Thibodeaux, et al., 1984; Eklund, et al., 1987). It should be noted that input to the model is highly dependent on the physical aspects of a particular CDF, the method of disposal, and the amount of time for a particular filling operation, as well as the lifetime of the CDF.

The equation used to calculate flux from exposed sediments describes chemical movement in the unsaturated pore spaces near the exposed surface. Although the initially placed sediments are in a semisaturated state, it is short-lived and surface layers will approximate the unsaturated situation soon after placement. In any case, this initial transient state is not accounted for by the model. Also, wetting and drying cycles generated by rainfall were not considered.

The major emission locales for a CDF and its inherent operations are as follows:

Dredging and transporting
Submerged sediments (ponded zone)
Exposed sediments void of vegetation
Sediments with vegetative cover

Due to complexities involved and lack of sufficient theory the model only considers the "submerged sediments" and the "exposed sediments void of vegetation" locales as emission sources for PCB flux.

## MODEL FORMULATION

Submerged Dredged Material (Pond Volatilization) Algorithms

The pathway for volatilization in the case of submerged dredged material involves desorption from the suspended solids phase, diffusion through the water, and transport through the airwater interface. Assuming a constant suspended solids concentration, the steady state flux of an organic chemical through the airwater interface is given by the following equation (Thibodeaux, 1988):

$$n_A = \frac{{}^1K_{A2}{}'}{K_d + 1/P_{32}} \frac{(W_A - P_{A2}{}^{**})}{1000} \quad (1) \tag{1}$$

where

| | | |
|---|---|---|
| A | = | organic chemical of interest |
| $n_A$ | = | flux of A through airwater interface, mg A/cm2 hr |
| $P_{A2}{}^{**}$ | = | hypothetical concentration in water for air side concentration of A, mg/L |
| ${}^1K_{A2}{}'$ | = | overall liquid phase mass transfer coefficient, cm/hr |
| WA | = | concentration of A in the original bed sediment, mg/kg |
| $P_{32}$ | = | concentration of suspended solids, kg/L |
| $K_d$ | = | sedimentwater distribution coefficient for A, L/kg |

With respect to the overall liquidphase mass transfer coefficient, when the emission rate is liquid phase resistance controlled, as it is for hydrophobic organics, ${}^1K_{A2}{}'$ depends on wind speed and

molecular diffusivity of A in water, and can be estimated using the following equation (Lunney, et al., 1985):

$$1K_{A2}' = 19.6 * V_x^{2.23} * D_{A2}^{2/3}$$ (2)

where

$V_x$ = wind speed, miles per hour (mph)
$D_{A2}$ = molecular diffusivity of A in water, cm /sec

If the diffusivity of A in water is not known, it can be estimated using the following equation (Thibodeaux, 1979):

$$D_{A2} = D_{B2}(M_B/M_A)^{0.6}$$ (3)

where

B = model organic chemical of known molecular diffusivity
$D_{B2}$ = molecular diffusivity of B in water, cm /sec
$M_A$ = molecular weight of A
$M_B$ = molecular weight of B

The quantity $W_A/(K_d + 1/P_{32})$ is the dissolved concentration of A in the pond water and can be thought of as the dissolved concentration of A at the water-air interface. The difference between it and $P_{A2}^{**}$ is the driving force which causes the flux of A into the air.

$P_{A2}^{**}$ is derived from the existing concentration of A in the air. This value is very small compared to the water concentration and therefore if assumed to be zero, it would have little effect on the driving force. This is a conservative assumption that maximizes volatilization.

Equilibrium partitioning uses the relative chemical solubilities of hydrophobic organic compounds (like PCBs) in sediment and water to estimate the concentrations of the compound in these two media .. equilibrium. PCBs are poorly soluble in water and have a high affinity for sediments, particularly those with much organic matter. The ratio of PCB concentrations in sediment and water at equilibrium is referred to as $K_d$. This partitioning coefficient ($K_d$) can be calculated from chemical properties of the contaminant (PCB) and information about the organic content (TOC) of the sediment or through a number of laboratory procedures. The $K_d$ for PCBs in the Indiana Harbor sediments was determined through laboratory procedures by the Waterways Experiment Station (WES) as 256,000 L/kg (USACE, 1987).

Equation 1 can be used for calculating flux from submerged sediments during filling. However, in between operations, the suspended solids concentration in the pond would not provide the driving force for PCB mass flux to the air since sediments containing sorbed PCBs, deposited during filling, settle rapidly after a filling operation stops. For this case, it is better to use the dissolved concentration in the water column as an indication of flux. The rate of mass flux across the phase boundary can be expressed by (Thibodeaux, 1979):

$$n_A = {}^1K_{A2}' (C_{A2} - C_{A2}^*)M_A$$ (4)

where

$C_{A2}$ = bulk liquid molar dissolved concentration of A, mol/cm
$C_{A2}^*$ = hypothetical concentration in water for air side concentration of A, mol/cm

## Exposed Dredged Material Algorithms

The volatilization pathway, in the case of exposed dredged material, incorporates a number of steps. Although sediments are placed in a semisaturated state, water and VOCs become quickly depleted from the surface layer, and continuing losses come from the pore spaces within the dredged material beneath the surface. At this point, VOC emission is dredged materialside vapor phase diffusion- controlled. The emission pathway involves desorption from particle surfaces into a water film surrounding the particles, diffusion through the water film, desorption from the water film into the pore gas, and diffusion through the pore gas prior to emerging into the atmosphere. This last step is apparently the limiting step in soil systems (Dupont, 1986), and this condition is thought to apply to the top layers of dredged material in a CDF (Thibodeaux, 1988). Fick's second law, with an effective diffusivity that accounts for tortuosity of the diffusion path and other factors that affect diffusion, is an appropriate mathematical model. Due to the depth of the dredged material and the relatively flat surface, a semiinfinite solution to Fick's second law can be applied without serious error. (The semi-infinite solution is conservative; that is, flux is maximized). The instantaneous flux is given by (Thibodeaux, 1988):

$$n_{A,t} = \frac{D_{A3}(E_1 + (K_d * P_B / H))}{\pi * t}^{1/2} \frac{(W_A * H - P_{A1i})}{1000 * K_d} \tag{5}$$

where

$E_1$ = air filled porosity, dimensionless
$D_{A3}$ = effective diffusivity, cm /sec
$P_B$ = bulk density of dredged material, kg/L
$H$ = Henry's Law constant, dimensionless
$P_{Ali}$ = background concentration in air at dredged material surface, usually assumed to be zero, mg/cm
$n_{A,t}$ = instantaneous flux of A through dredged materialair interface at time t, mg A /cm sec
$t$ = time since initial exposure, sec

The average flux over a given time t is given by

$$\bar{n}_A = \frac{\int_0^t n_A dt}{\int_0^t dt} \tag{6}$$

It can be shown that

$$\bar{n}_A = 2n_{A.} \tag{7}$$

The above equation is an idealized diffusion transport model that describes chemical movement in the unsaturated pore spaces near the surface of exposed dredged material. It does not account for the development of cracks as the dredged material de-waters by evaporative drying.

Effective diffusivity is a constant diffusion coefficient that characterizes the movement of chemical A as a vapor within the porous solid. It is one parameter for which there is no information available. To calculate the flux, it is therefore necessary to estimate DA3. As an approximation, tortuosity can be accounted for using the equation below (Thibdodeaux, 1988):

$$D_{A3} = \frac{D_{A1} (E_1^{10/3})}{E^2} \tag{8}$$

where

$D_{A1}$ = molecular diffusivity of chemical A in air, cm /sec
$E$ = total porosity, dimensionless

Henry's law constant (H) applies for dilute solutions of chemicals in air and water. It is an equilibrium partition coefficient for chemical A between the air and water phase. Henry's Law constant can be estimated using the equation below (Dilling, 1977):

$$H = \frac{16.04(P_A^\circ * M_A)}{T * P_{A2}^*} \tag{9}$$

where

$P_A^\circ$ = vapor pressure of A as pure solute, mm Hg
$P_{A2}^*$ = solubility of A in pure water, mg/l
$T$ = temperature, degree Kelvin

The background concentration Pai in air has an analogous meaning to $P_{A2}^{**}$ and also is assumed to be zero. This is a conservative assumption that maximizes volatilization.

## RESULTS

In comparison, the inlake CDF showed the least amount of volatilization. This is because over the filling life of the CDF the exposed surface area is much less than an upland CDF. During most of the filling, the dredgings are placed and remain submerged. Table 1 shows the maximum annual simulated PCB loss for three contaminant transfer pathways.

Table 1. Maximum Annual PCB Loss (Lbs)

| In-lake CDF | | | Upland CDF | | |
|---|---|---|---|---|---|
| Seepage | Leachate | Volatile | Seepage | Leachate | Volatile |
| 0.0001 | 0.001 | 2 | 0.0001 | 0.001 | 8 |

## INTERPRETATION OF RESULTS

The results indicate that volatilization of VOCs is a significant contaminant transfer pathway. Also, PCB mass flux is less when the sediments are maintained in a submerged state. This is due to the hydrophobic nature of PCBs. Also, the flux is highly dependent on two factors; the exposure time of the sediments, and the surface area of the sediments. The exposure time for submerged sediments encompasses the entire time a pond is in contact with PCB contaminated sediments. However, the rate of volatilization is directly related to the concentration of dissolved PCBs in the pond, which is derived from the mass fraction of PCBs in the sediments. The rate of volatilization changes over time, since the pond dissolved concentration of PCBs varies over time with the highest rate during an active filling operation. The surface area is that area of the pond which is in direct contact with the air and is dependent on the volume of dredgings being deposited and the volume of material already placed within the CDF.

The exposure time for exposed sediments encompasses the time in which unsaturated sediments are in direct contact with the air, while the surface area is that area which is in direct contact at any given time.

In summary, the approach taken in model formulation was conservative in nature in that it simulated a worst case scenario. For instance, the exposed sediments were assumed to be completely void of vegetation throughout the life of the CDF. However, from past experience, a vegetative cover will form over the exposed sediments over time. There is no quantitative theory predicting the effects of vegetation on flux. However, it is anticipated that the vegetation cover would reduce the flux rate. Also, the surface area of exposed sediments was simulated as a layer covering the entire cell (only for upland CDFs). Realistically, the deposited sediments would flow outward, but probably not far enough to cover the entire cell of an upland CDF.

Finally, the suspended and/or dissolved solids concentrations in the ponded areas were based on conservative estimates. For the reasons stated above, the actual PCB mass flux from a CDF could be substantially lower then what is predicted by the model simulation.

## CONCLUSIONS

Theoretical models must be tested against and adjusted to both laboratory and field data prior to their acceptance and widespread use as predictive tools. Preliminary model calculations can be made for the submerged sediment locale and the exposed sediment locale void of vegetation. However, some aspects are based on very crude equations and further development is needed. At this time, laboratory and field testing must be performed to build a higher degree of confidence in the predictive capability of the PCB volatilization model.

Laboratory analysis has recently been completed by the Waterways Experiment Station (WES) on New Bedford Harbor sediments in order to determine the volatile emission rates of PCBs from freshly placed drying sediments (USACE, 1989). This experiment was ran under laminar conditions, not accounting for wind. These conditions represent an overall simplified condition but do support the theory presented in this paper. In any event a substantial amount of work in lab/field testing and verification needs to be completed before any conclusive results can be made on PCB flux simulation from an active CDF.

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# ESTIMATING COSTS OF SEDIMENT REMEDIATION TECHNOLOGIES

by

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## INTRODUCTION

The Chicago District has conducted a detailed analysis of the technical, environmental, and economic feasibility of applying treatment technologies to contaminated dredged materials from Federal navigation channels. This analysis was performed as a result of requests from the citizens of northwest Indiana during the preparation of a Draft EIS for dredging and disposal activities at Indiana Harbor. This paper summarizes that analysis and pays particular attention to the economic analysis of the application of treatment technologies as a dredged material disposal alternative.

## HISTORY

With the improvement in water quality throughout the Great Lakes since the early 1970s, the public has turned its attention to the issue of contaminated bottom sediments. Environmental groups, spurred by their involvement in the development of Remedial Action Plans for the 42 Areas of Concern on the Great Lakes, have made sediment remediation a top priority, and have placed particular emphasis on the application of advanced technologies from the field of hazardous waste treatment to contaminated sediments. Some groups have even proposed a new mission for the Corps, making it responsible for the remediation of contaminated sediments both within and outside of Federal navigation projects.

Although the treatment of contaminated dredged materials is a current topic, the problem is not new to the Corps. The 1969 report, Dredging and Water Quality Problems on the Great Lakes, explored alternatives to the open-water disposal of contaminants that included treatment technologies. Some of the treatment technologies were tested on a bench or pilot scale, including: pumping the dredged materials to a wastewater treatment plant; aerobic and anaerobic degradation; chemical oxidation through addition of chlorine; and wet air oxidation.

With the passage of PL 91-611, Congress authorized the Dredged Material Research Program (DMRP) to expand and refine the technical base developed through the Dredging and Water Quality on the Great Lakes report. Through Research Task 6B, the DMRP specifically addressed the issue of treatment of contaminated dredged material.

The Corps has also evaluated using dredged materials for a number of beneficial applications. These uses included reclamation of strip mined areas, making bricks, mechanically densifying the material, drying the material, and using the material as a sanitary landfill cover. Some of these applications were tried by the Chicago District and others, most notably the Waterways Experiment Station. All of these particular techniques hold promise for use with cleaner dredged materials, but the use of contaminated materials like those in the Great Lakes has not been encouraged.

Any economic analysis of dredged material disposal alternatives must take into consideration the operational constraints of the Corps. The goals to be met by a Corps of Engineers maintenance activity, such as dredging contaminated sediments from a Federal navigation project, are stated in 33 CFR 335.4.

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"The Corps of Engineers undertakes operations and maintenance activities where appropriate and environmentally acceptable. All practicable and reasonable alternatives are fully considered on an equal basis. This includes the discharge of dredged or fill material into waters of the U.S. or ocean waters in the least costly and most practicable location, and consistent with engineering and environmental requirements." The alternative that meets these criteria is called the "Federal Standard."

## TECHNOLOGY TYPES

The technologies being considered for treatment of contaminated dredged materials come from a variety of sources, primarily the hazardous waste treatment arena, but also include modified processes from the mineral recovery and sewage sludge treatment fields. This section will provide a brief description of the main types of processes: degradation, extraction, immobilization, and immobilization/degradation.

Degradation technologies are based on thermal, chemical, or biological processes that act to alter the chemical form or properties of contaminants. Degradation technologies are capable of breaking down organic contaminants into a mixture of simple molecules and intermediates that are less toxic or non-toxic, but they cannot degrade metals.

Extraction is the removal of chemical constituents from contaminated material with the goal of producing an uncontaminated residue. Extraction technologies use physical and chemical processes to transfer contaminants to another medium, generally a fluid, for treatment and disposal by another set of processes. Sometimes treatment of organic contaminants after extraction can be carried out under more favorable conditions, at lower risk, and at reduced costs.

Immobilization is physical, chemical, or combined physical/chemical conversion of contaminated material and waste constituents to a form that is resistant to leaching, erosion, biological attack, and other transport process responsible for movement of contaminants in the environment.

Immobilization/degradation technologies are primarily designed to immobilize contaminants but because of unusual aspects of waste processing they also degrade and remove some organic contaminants during the immobilization process.

## ESTIMATING COSTS OF ADVANCED TECHNOLOGIES

This section on cost estimates must be prefaced with a caveat and discussion of the different elements presented herein. A detailed cost estimate of this type, for an entire sediment remediation process, has not been attempted for a project of the magnitudes being considered in the Great Lakes (millions of cubic yards). Cost estimates produced for U.S. Environmental Protection Agency (USEPA) at hazardous and toxic waste cleanup projects where similar treatment technologies are used typically have an acceptable accuracy of plus or minus 50 percent. The cost estimates provided in this appendix were developed by the Chicago District and are within this acceptable range of plus or minus 50 percent.

The accuracy of the overall cost estimates depends entirely on the accuracy of the estimates developed for each individual element. The different elements that make up a complete dredging and disposal process are discussed below, and it should be understood that for some of these elements, our cost estimating abilities are excellent, due primarily to experience with such operations. For other elements, where little or no experience exists, estimates must be developed based on the best engineering judgment of the cost estimator.

The process of removing contaminated sediments from a waterway and their subsequent treatment and final disposal can be broken down into four basic steps:

a. Dredging
b. Pre-treatment

c. Treatment technology
d. Post-treatment

Each of these four steps is absolutely necessary in any treatment process, and each of these steps has costs that impact the overall cost of the project.

Of the four steps identified above as being part of a treatment process, the costs associated with dredging are the best known and most easily identifiable. This is primarily due to the extensive experience developed by the Corps over the past 100 years of navigation maintenance. The cost of dredging is affected by four variables: the volume and pollution level of the material to be removed, the accessibility of the material, the distance to be traveled during dredging and disposal, and any restrictions that may be placed on the dredging.

As with the costs associated with dredging, the costs for the pre-treatment of the dredged materials are fairly well known. This is because the processes that may be used to prepare the dredged materials for treatment are existing handling techniques that have been used for manipulating materials similar to dredged materials, such as sludges, waste slags, or soils. A certain amount of uncertainty arises from the application of these processes to dredged materials, which have unique physical and chemical properties. The unit costs for the pre-treatment of dredged material are affected by storage requirements, the amount of dewatering required, any debris removal, and the rehandling and transportation of materials that may be required.

Dredging can be performed most efficiently at a pace that exceeds the rate that the materials can be treated by any of the technologies. A storage facility will have to be constructed to stockpile the dredged materials as they are dredged and before they can be treated. The costs for building confined disposal facilities, which are this type of structure, are well known.

The costs associated with the construction and operation of the individual treatment technologies are the least well known when compared to the costs of dredging, pre-treatment and post-treatment, because none of the technologies under consideration have ever been executed on the scales now being considered in the Great Lakes. Experience from the areas of hazardous waste remediation and from traditional civil engineering construction projects can be used in developing the unit costs for the different technologies. These unit costs are affected by the complexity of the process, the size of the physical plant required, the amount and chemical nature of any additives or reagents used, the controls required for any process discharges, the amount of energy used, the size of the labor force needed, and any safety precautions necessary.

None of the treatment technologies under consideration can completely destroy all of the contaminants present in dredged material. All of the technologies will produce one or more end products that will have to be disposed of or require further treatment before being disposed or discharged. The unit cost for post-treatment will depend on the amount of water treatment required, any air or volatile treatment necessary, the final disposal of the solid material that will always be left over, the treatment of any other fractions or residues that may be produced, and any rehandling or transportation of these end products that must occur.

## EXAMPLE TREATMENT PROCESS

From the discussion above, it can be seen that dependent on the treatment technology selected, the number and type of supporting processes needed will vary. This section details one possible process centered on the use of a solvent extraction technology that could be employed to treat the sediments from a Great Lakes harbor. The goal of this hypothetical process is to treat the contaminated materials as complete as possible. Ten steps make up the example process. The steps, from the initial removal of materials from the waterway to the final disposal of end products, are described below.

## Dredge material

Mechanical dredges can remove sediments from the waterway at or very near their in-place water content, unlike hydraulic dredges, which add up to four times as much water to form a slurry. This would greatly reduce the amount of dewatering that must occur during storage. The lower water content of the mechanically dredged material will reduce the amount of drainage water collected, and the amount of water that must be treated on-site. Mechanical dredging will be used.

## Store material in CDF

This step is necessary because dredging can be most efficiently performed at a rate that exceeds the treatment rate of the solvent extraction technology. With mechanical dredging being used, the CDF size will be smaller than would be used for storing hydraulically dredged materials.

## Remove large debris (>12")

This step actually takes place partly during the rehandling of material from the scows and into the CDF, when the unloading crane can be used to separate very large debris (timbers, rocks, car bodies) from the bulk of the dredged material. Further screening of large debris can be done after the materials have consolidated and before, or as, they are removed from the CDF.

## Remove medium sized debris (>2")

In addition to removing all large debris, the de-watered dredged materials will be screened to remove any remaining debris larger than 2 inches. Items such as rocks or waste iron scraps could damage the reactors used by the extraction process.

## De-water material through underdrain system

This step is inherent in the operation of any CDF. As the materials are allowed to consolidate during storage in the CDF, the interstitial water that drains from them will be collected in a series of underdrain pipes for treatment and discharge.

## Rehandle material into extraction unit

This process is centered on an on-site extraction plant, built for the extraction of organics from contaminated sediments. Rehandling will involve the removal of the dried dredged material from the CDF by standard earthmoving equipment either into trucks or onto a conveyor system for transport to the on-site extraction reactor.

## Extract material

The extraction plant built on-site will employ the BEST technology, a proprietary process using triethylamine as the solvent. A plant will be built on-site capable of extracting 292 cubic yards of dredged materials per day (about 250,000 cubic yards per year). The plant will produce three fractions of material (oil, solids, and water) that will be disposed of as described below.

## Incinerate oil fraction

If we assume the sediments to be treated have an average oil content of 6.4 percent (typical for sediments from heavily polluted Great Lakes harbors), at a processing rate of 292 cubic yards per day of dredged material the extraction plant will produce 3800 gallons per day of organic-contaminated oil. This oil fraction will be placed into tank trucks and transported off-site where it will be incinerated.

## Rehandle solid fraction into disposal site

The solid fraction from the extraction plant will be solidified with portland cement and/or additives and disposed of in the DF originally used as the storage and dewatering facility. Rehandling will involve moving the material either by truck or conveyor from the solidification plant to the CDF. The CDF will be capped after filling.

## Treat effluent and drainage water

The drainage water collected from the CDF during storage of the dredged materials will require some form of treatment. The extraction plant will produce a water fraction that will also require treatment before disposal. The treatment of both the drainage water and the extracted water will be accomplished with an on-site wastewater treatment plant.

## UNIT COST ESTIMATES

This section presents cost estimates for each of the elements identified above, and sums them to produce a unit cost for the treatment technology process evaluated. As discussed above, the cost estimate on some items, such as dredging, transport, and CDF construction, may be very accurate, due primarily to the extensive experience in constructing these types of facilities the Corps has gained in the past 100 years. On the opposite side, most treatment processes have never been attempted on either the scale or for the type of materials now under consideration at Great Lakes harbors. In developing these estimates we have attempted to use a mid-range figure for the costs. All of these unit costs include both the capital cost required to build the necessary structures and facilities as well as the operation and maintenance costs that will be incurred as they are in use. The unit costs for each of the elements in the example treatment process are:

|  |  | $ per cu. yd. |
|---|---|---|
| 1. | Dredge material | 6.00 |
| 2. | Store material in CDF | 10.00 |
| 3. | Remove large debris (>12") | 2.00 |
| 4. | Remove medium sized debris (>2") | 2.00 |
| 5. | De-water material through underdrain system | 1.00 |
| 6. | Rehandle material into extraction unit | 2.00 |
| 7. | Extract material | 90.00 |
| 8. | Incinerate oil fraction | 40.00 |
| 9. | Rehandle solid fraction into disposal site | 2.00 |
| 10. | Treat effluent and drainage water | 5.00 |
|  | Subtotal | 158.00 |

The costs given above for the treatment technology process were summed as a subtotal, indicating that there are other costs in addition to those directly related to the treatment technology that must be considered. These costs, calculated as a percentage of the subtotal, are those associated with ordinary engineering or construction projects, and are shown on Table 1.

Table 1: Additional Costs for Treatment Technology Process

| Cost Item | Percentage of Subtotal |
|---|---|
| Mobilization/Demobilization | 25 |
| Engineering & Design | 15 |
| Supervision & Administration | 8 |
| Process Development | 7 |
| Contingency | 25 |

Mobilization/demobilization is the cost incurred during the transportation of necessary equipment to the project site and the setup and breakdown of the equipment. Some equipment may become contaminated during operation and require special handling and disposal after breakdown. Engineering and design is the cost of the design work that will go into the plans for all of the components of the process the treatment plants, the storage facility, the treatment operation, etc. The eight percent allocated for supervision and administration is a fixed percentage for Corps projects and covers the costs for the personnel who will supervise the overall project to assure that the operation is conducted in accordance with the specifications and administer the contracts.

The final two costs, process development and the contingency fee, are the two least well-known of all the costs discussed to date. The costs that will be incurred for process development are very dependent on the specific process being used and the current state of development of that process. This cost will be significant for any technology because they all will need to be scaled up to sizes they have never been operated at before. Extensive testing would also have to be done with the specific sediments identified for treatment to determine the proper amounts of reagents and chemicals necessary to make the technology work. The contingency fee, which is set at 25 percent, is an attempt to make allowances for any of the other costs being grossly in error. It does not guarantee, however, that the actual costs will be below the estimated total cost. The total unit cost for this example treatment process is given in Table 2.

Table 2: Total Unit Cost for Example Treatment Process

| Item | Cost ($/cy) |
|---|---|
| Subtotal (from Table 1) | 158.00 |
| Mobilization/Demobilization | 39.00 |
| Engineering & Design | 23.70 |
| Supervision & Administration | 12.64 |
| Process Development | 11.06 |
| Contingency | 39.50 |
| Total | 284.40 |

## SUMMARY AND CONCLUSIONS

Cost estimates were developed for a hypothetical sediment remediation process that was designed to completely treat the contaminated sediments from a heavily polluted Great Lakes harbor. This cost estimation exercise includes the complete costs of remediation, from the removal of the material from the waterway by dredging through the final disposal of the treated end products. The total unit cost for the process was estimated at $284.40 per cubic yard of material, and is accurate to within plus or minus 50 percent.

In summary, the entire contaminated sediment remediation process must be seen in terms of improving the overall quality of the aquatic ecosystem. The addition of "treatment" to a disposal process will not make environmental impacts go away. Treatment technology applications will have many of the same impacts associated with confined disposal alternatives. Some new impacts may be created by these technologies. The environmental impacts of advanced treatment technologies with dredged sediments have not been fully assessed in any studies completed to date.

The underlying reason for considering advanced treatment of contaminated sediments is to improve the environmental performance of their disposal. This might be done either through the destruction and removal of contaminants of concern or by altering the material to make their disposal more efficient. Advanced treatment of these sediments is not required under Federal law and is appropriate only if found to be the least costly alternative that is consistent with engineering and environmental requirements.

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# DISPOSAL OPTIONS FOR DREDGED MATERIAL
## IN THE DETROIT DISTRICT

by

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## INTRODUCTION

The Detroit District of the Corps of Engineers is comprised of over 100 navigation projects located on the Great Lakes in the states of Michigan, Minnesota and Wisconsin. Eighty-four of these projects are actively maintained under the Corps' Operation and Maintenance program. The Detroit District dredges 1.75 million cubic yards of shoaled material each year and an extensive sediment and water quality monitoring program is required to evaluate the quality of material to be dredged and to estimate water quality impacts.

Prior to authorization to construct Confined Disposal Facility (CDFs), dredged material was mainly deposited in deep water sites. Several projects, however, were developed to utilize dredged material in habitat development. Since 1970 twenty-six CDFs have been constructed upland, in-water, and nearshore within the Detroit District. CDFs have been constructed, under Public Law 91-611, to contain contaminated dredged material at 15 of these sites. This legislation mandated the construction of CDFs to contain 10 years worth of dredged material.

## SEDIMENT SAMPLING

The District routinely conducts sampling and analysis of District harbors in order to evaluate potential changes in disposal options for dredged material. Seventeen of the Detroit District projects encompass regions designated as Areas of Concern (AOCs) by the International Joint Commission. The States and Provinces have been charged with developing Remedial Action Plans (RAPs) following Commission guidelines to identify specific actions needed to restore beneficial uses to these areas. The information developed under the District's sediment monitoring program is important for the RAPs to assess in-place pollutants, develop mass balances and address remedial costs and the needs of RAPs are considered in the development of sediment sampling projects.

Small shoals or shoaling in areas characterized by littoral drift are sampled using ponar grabs. More typical sampling of unconsolidated material is completed using a gravity coring device. Driven cores are collected for deepening projects or in situations where deferred maintenance has caused heavy shoaling.

Sediment samples are routinely analyzed for 10-12 metals, 7 PCB mixtures, 20 other chlorinated compounds as well as non-specific indicators such as chemical oxygen demand, total Kjeldahl Nitrogen, oil and grease, etc. Base/Neutral/Acid extractables and Volatile Organics have been surveyed at some of the Areas of Concern to develop a baseline, to assist RAPs, and to determine if operational requirements may be impacted. Evidence to date has shown a wide variation in Polynuclear Aromatic Hydrocarbon concentrations.

However, minimal impacts on dredging operations are foreseen. Elutriate and background water testing is conducted in situations where open-water disposal or beach nourishment is contemplated. Contaminant concentrations are evaluated in comparison to background concentrations at the disposal site and using 1977 Guidelines promulgated by the Environmental Protection Agency (EPA).

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Two example projects conducted in coordination with RAPs are Milwaukee, Wisconsin, and Manistique, Michigan. At Milwaukee, the District collected additional volumes of sediment at each sampling site. The additional sample was provided to the RAP coordinator who was able to arrange bio-uptake testing of the material. In addition, an aliquot of the samples will be tested by the Wisconsin Department of Natural Resources for exotic contaminants outside the realm of typical Corps O&M monitoring. At Manistique, Michigan PCB concentrations exceeding 50 ppm are effectively preventing maintenance dredging. In this instance, the Michigan Department of Natural Resources and the Corps will complete a joint sampling effort where the Corps will sample the Federal project area and the State will sample the area outside the Federal project. Both joint efforts should provide a comprehensive survey of contaminant concentrations, throughout the AOC.

The District is in the process of developing a Geographic Information System (GIS) for handling the large amount of sediment data generated with each project. Chemical data generated by the contract laboratory are submitted in a bound report and on a diskette in a dBase III format provided by the District. From the dBase file the data can be translated to STORET format and to a format compatible with the GIS. The GIS is configured with a Compaq 386/25 personal computer with SPANS software which allows mapping of contaminant concentrations. Maps can then be overlain to allow viewing of multiple maps at one time. Plans include networking to the Michigan Resources Information System (MIRIS) which will provide information on wastewater discharges, known contamination sites, water intakes, fish spawning areas, etc.

## BENEFICIAL USES OF DREDGED MATERIAL

The Detroit District makes every effort possible to obtain the maximum practicable benefit when disposing of dredged material. Each project is tailored to accommodate the particular needs and logistics at each harbor and the physical and chemical characteristics of the material. Beneficial use projects which have been developed include: (1) beach nourishment/shoreline stabilization; (2) habitat development; (3) agriculture; (4) construction, industrial and development use; and (5) road sanding in the winter.

### Beach Nourishment

Beach nourishment has been conducted at 27 harbors within the Detroit District. It is a low cost, beneficial option for many of the dredging projects and is considered whenever sediments are not contaminated and particles sizes indicate a coarse sediment. When developing dredging plans for a particular project, areas of erosion are considered along with distance from the dredging areas which directly affects the cost of the operations. Locations of state, city, and county parks and the status of the erosion along them, and other water features such as water intakes are also considered.

The method of dredged material placement varies among projects. In some cases, material is pumped directly on the beach or at the shoreline and spread lakeward to enhance the existing beach. In other cases, the material is deposited from the Ordinary High Water Mark to a specified contour (usually the 4- or 8-foot contour) and may be hydraulically pumped or bottom dumped. Another method of placement is bottom dumping of the material at the 8-or 12-foot contour. All of these methods provide stabilization to the shoreline by direct enhancement or by reducing the wave energy in the nearshore area.

Under Section 111 of the River and Harbor Act of 1968, the Corps of Engineers is authorized to conduct beach erosion mitigation. Section 111 provides for the mitigation of shore damage attributed to Federal navigation structures but does not provide for protection from natural erosion. Projects are evaluated and, if it is determined that erosion has occurred due to the interruption of littoral drift by the Federal navigation structures, a plan is developed to provide relief to the problem areas. Fourteen such projects have been authorized in the Detroit District. In most cases, clean dredged material from the outer portion of a channel is placed in defined areas of erosion for mitigation purposes. Dredged material is sometimes supplemented with material from a borrow area or an upland source to meet the

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annual erosion attributed to the Federal navigation structures. Monitoring is conducted at the Section 111 projects to evaluate the mitigation activities. Where necessary, mitigation activities are modified.

Habitat Development/Island Creation

Confined Disposal Facilities develop various habitats during the filling process, depending on the construction site (aquatic, wetland, upland), although in most cases the final state of the CDF will be upland habitat. These sites provide additional upland habitat as well as recreational opportunities when completed. In urban areas the upland habitat created is a unique resource. Many of these disposal sites serve as a stopover for migratory birds and provide nesting areas for several bird species. The island CDFs may provide an isolated habitat free of ground-dwelling predators which is not otherwise available in urban areas.

Of special mention is the Pointe Mouillee CDF designed to contain contaminated dredged material from the Detroit and Rouge Rivers. The Pointe Mouillee marsh is located on the western shore of Lake Erie, below the mouth of the Detroit River and covers an area of about 2,600 acres. The Pointe Mouillee State Game Area is managed by the State of Michigan primarily for waterfowl and is a major stopover point for both fall and spring migrants.

The marsh has been subject to severe inundation and significant erosion by high water and waves during the last four decades. Loss of a natural barrier beach resulted in serious erosion to the marsh. A CDF was constructed which is crescent shaped, 3.5 miles long and 1,400 feet wide with its western limit along the area of the previous barrier beach. The facility provides a protective barrier against wave erosion to the existing marsh. The Michigan Department of Natural Resources is managing water levels behind the CDF in an effort to re-establish the eroded marsh. This area is divided into several units and pumping facilities will be added in the future which will allow management of water levels. The CDF covers an area of approximately 685 acres and has a capacity of 18,640,000 cubic yards. The facility also provides access to the lake for the public.

Following are a few more examples of island creation and habitat development projects: (1) Grassy Island CDF - Grassy Island is located in the Detroit River and provides upland habitat in an industrial area of Detroit; (2) Gull Island - Gull Island is located in Lake St. Clair at the mouth of the St. Clair River. The island was created from sandy dredged material from the St. Clair River and is a popular swimming/recreational area for boaters; (3) St. Mary's River Islands - Rock and sandy material removed from the St. Mary's River was placed adjacent to the Federal Channel to enhance existing islands and create several new islands which provide breeding habitat for waterfowl; (4) Sterling State Park CDF - The Sterling State Park CDF was developed on land in a state park which is the third busiest in the state. When the CDF is filled, the area will be opened to the public as a recreational area. The State intends to construct a nature center on the CDF.

Agriculture

Dredged material with excessive nutrients from Frankfort Harbor, Michigan was utilized to reclaim land for farming purposes. The landowner planned to develop orchards over the reclaimed 20 acres.

A project currently under design includes use of nutrient rich sediment from Sturgeon Bay Harbor, Wisconsin as fill for a tree nursery.

Construction, Industrial, and Development Use

Several projects in the Detroit District have utilized dredged material in construction, industrial or development use. These include: fill, dike construction, urban and residential use, parking lots, roads, road sanding, etc.

One recent success involves a sediment washing project conducted at Duluth-Superior Harbor. The Erie Pier CDF is nearing capacity and the District is attempting to extend its lifespan while developing a new site. The bidder developed a Value Engineering Proposal which involved placing the dredged material in a ponded area within the CDF and agitating the dredged material using earthmoving equipment. Agitation suspends the fine sediments which are then carried down-gradient to a settling pond. The only material which can settle in the washing area is coarse sandy material. A crane then removes this settled material, which is now relatively free of contaminants and fine material. Water from the settling pond is then recirculated through the washing procedure again. The washed material is allowed to dry on-site and may be used for road fill or in construction.

Samples of the dredged material were found to contain 70% fines while the washed material contained 15% fines. Cadmium and lead were reduced by 70% and 80%, respectively. PCBs averaged 0.10 ppm in the dredged material, but were not detected at a detection limit of .02 ppm in the washed material.

Another project under development at the Black River at Port Huron, Michigan would use dredged material as a final cap for a Superfund site. The State is treating contaminated soil at the site to remove as much contamination as possible and proposes to cap the entire site with dredged material.

## OPERATIONAL CONSIDERATIONS OF CONFINED DISPOSAL FACILITIES

### Design

The CDFs were constructed both in-water and on-land; the dike walls consist of clay, prepared limestone, or rock and sand mixtures. Weir structures were required at most the CDFs to allow for release of excess water pumped in by hydraulic dredging. The discharge of dredged material into the site is located away from the weir and a sufficient amount of retention time allows for solids to settle and result in an acceptable effluent discharge.

### Water Quality Monitoring

Water quality monitoring is performed to provide an indication of the environmental effects of our activities and subsequently provide a check on dredging disposal operations to ensure environmental acceptability. The parameters to be monitored are based on sediment data of the channel material. The types of facilities monitored include: (1) CDFs with a weir outflow with impermeable dikes (i.e., with a clay core) and permeable dikes (i.e., rock and sand mixtures or prepared limestone); and (2) CDFs without a weir outflow which have a filtered release. These are generally constructed with a prepared limestone core. Effluent release is through the dike wall itself or through filter cells.

The water quality monitoring stations for a CDF with a weir overflow into a lake include: (1) dredge discharge; (2) disposal facility weir overflow; (3) mixing zone; and (4) three ambient stations located in a semicircle 250 feet from the weir discharge. The water quality monitoring stations for a CDF with a weir overflow into a river include: (1) dredge discharge; (2) disposal facility weir overflow; (3) mixing zone; (4) upstream (background); and (5) downstream.

The monitoring stations for a CDF with a filtered release would generally be the same, except for the absence of a weir sample, a sample would be obtained from the ponded water inside the CDF near the filter cells, and from the monitoring well located in the filter cell.

The frequency of monitoring is based mainly on the remaining capacity of the facility and pumping rate of the dredge since these factors are most directly related to retention time and quality of effluent. The duration of disposal and known problem contaminants which must be given specific attention are also used to determine the frequency of testing.

## Botulism Prevention

Colonies of terns, gulls, and/or herons typically nest on CDFs, which provide unique habitat in urban areas for wildlife use. The presence of mud flats and shallow ponded areas can lead to the development of avian botulism. Since these sites are so actively visited by waterfowl, the Detroit District has developed management plans to ensure that placed material is graded such that the slope is towards the outlet structures. Proper filling techniques ensure drainage of the CDF through an outlet structure. A second approach to prevention of botulism is to schedule the dredging operation during the cooler seasons when the potential for an outbreak is minimized. However, adverse weather conditions in the spring and fall limit available time required for dredging.

Regular inspections of disposal sites are conducted during the warmer seasons to determine if conditions for a botulism outbreak are present. If problems are found, the reaction phase is initiated and includes collecting and removing dead or sick birds, activating exploders, and/or flooding the dike interior to cover mudflats. There have been three incidents of botulism at Detroit District CDFs, the highest loss was about 525 birds in 1980.

# COMPUTERIZATION OF THE DECISION MAKING FRAMEWORK

by

Charles H. Lutz[1], Victor A. McFarland and Bobby L. Folsom, Jr.[2]

## INTRODUCTION

The general Decision Making Framework (DMF) for management of dredged material (Peddicord et al. 1986) was developed to provide a systematic approach for selecting the best option for disposal of dredged sediments based on environmental concerns, cost, and site availability. Using a tiered testing approach, the DMF first guides the user to a decision whether there is "reason to believe" sediment contamination may require disposal restrictions. The tiered approach allows the necessary and sufficient level of testing to be used for each specific project. In Tier I testing, existing information and simple, inexpensive procedures are used to determine potential environmental impact. Later tiers are implemented if the initial results show a potential for impact or if they are inconclusive. This minimizes excessive testing and results in more efficient completion of required evaluations and reduced costs (Environmental Protection Agency/Corps of Engineers, 1989).

If "reason to believe" exists, the DMF then identifies the data requirements necessary for further decision making regarding suitability of the material for the chosen disposal option. The user provides data from chemical analysis of the sediments proposed for disposal and from reference sediments, data from bioassays and bioaccumulation evaluations, and data describing the physicochemical nature of disposal site environments. Decision-enabling criteria that are locally determined or are provided by national standards are applied to the input data using a series of flow charts that lead to the most environmentally desirable disposal option.

The Environmental Laboratory at the Waterways Experiment Station (WES) is preparing computer software that will greatly increase the speed and ease of application of the DMF for those involved in the management of dredged material. The DMF generalized flow chart is illustrated in Figure 1. Each block is subdivided into secondary flowcharts that implement detailed areas of the DMF. Figure 2 shows the aquatic disposal option of the DMF in more detail. Computerizing the DMF procedure frees the user from separate data recordkeeping, mathematical calculations and comparisons. Using predictive models, data from the literature and data entered or selected by the user, the DMF computer program will provide the user with summaries and guidelines to assist in making appropriate dredged material disposal decisions. Once the necessary data have been entered into the program, results may be obtained in minutes instead of hours or days.

The current computerization effort covers Tier I testing ("reason to believe," and the comparison of test and reference sediments) and partial Tier II testing; but more importantly, it provides a "front end" and uniform data entry platform for the integration of the test modules. These modules implement the various testing protocols needed to determine the applicability of available disposal options. The test modules are being developed under various other programs.

## HARDWARE/SOFTWARE REQUIREMENTS

The DMF software was developed on industry-standard 80286 IBM PC AT compatible microcomputers with 640 kilobytes of memory, a 20-megabyte hard drive and an enhanced graphics

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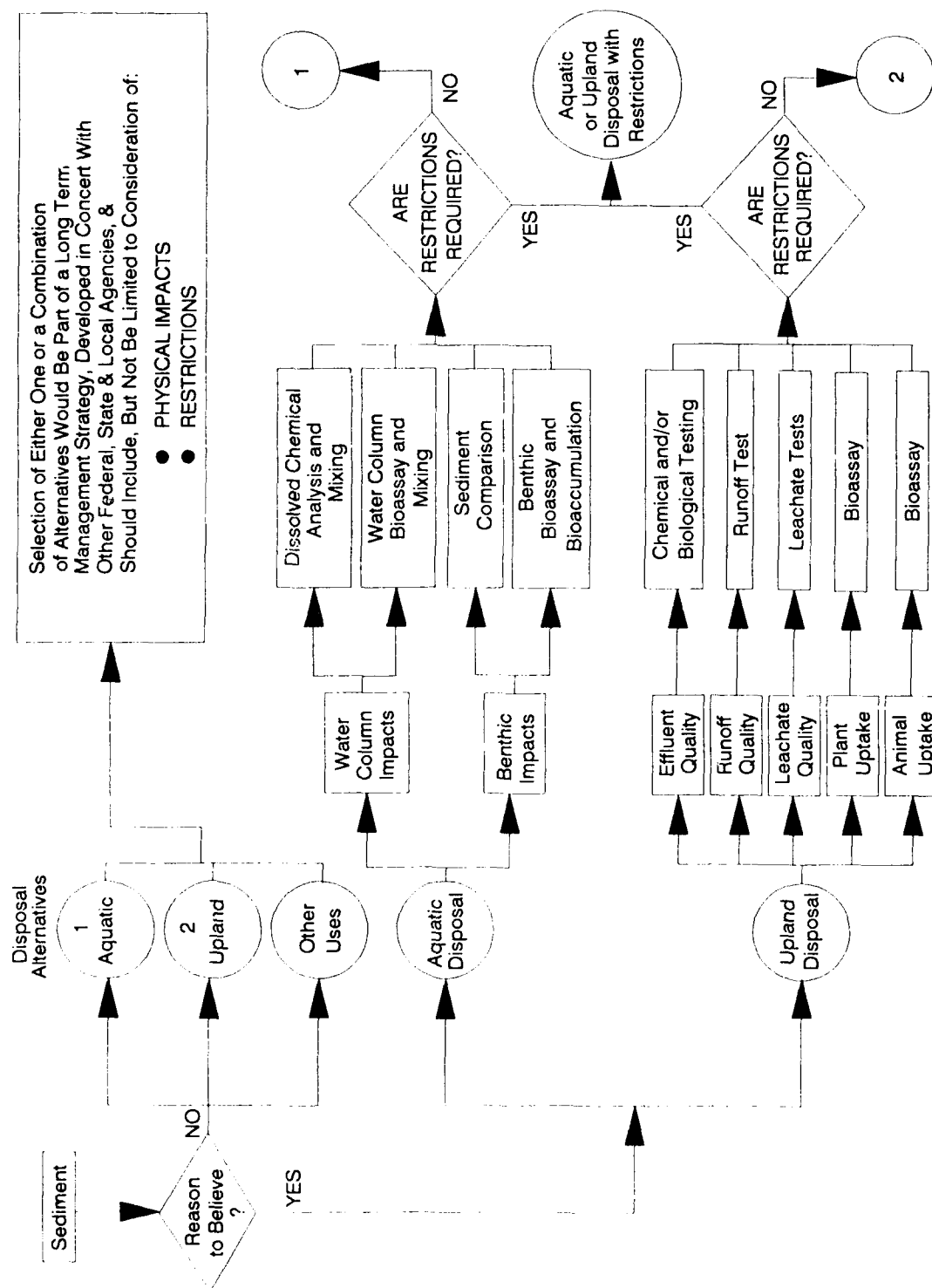
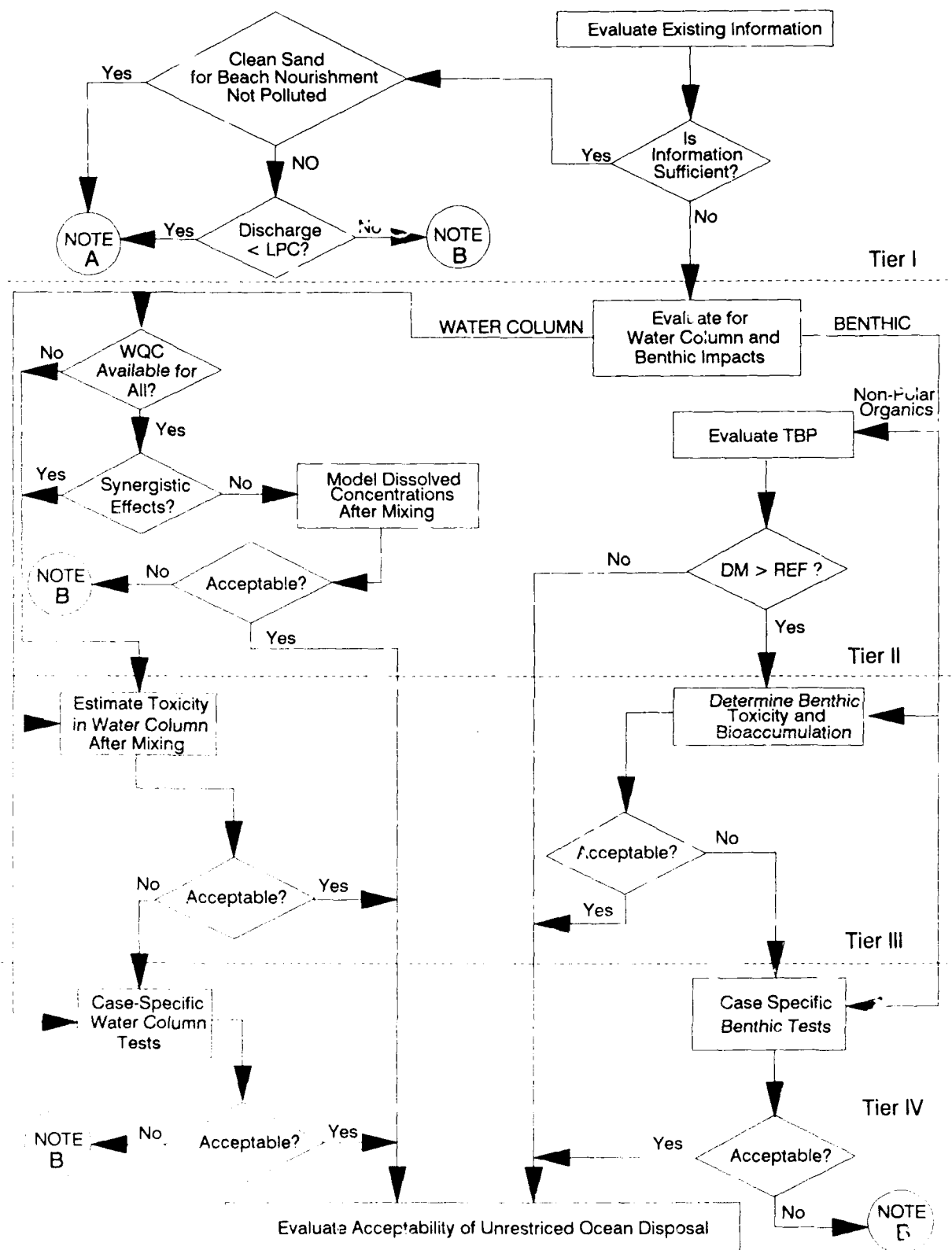Figure 1. DMF Flowchart. Modified from Peddicord et. al. 1986.

Figure 2. Aquatic Disposal Flowchart. (Modified from EPA/COE 1989)
See Appendix I for Notes and Abbreviations.

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(EGA) color monitor.  This configuration is generally available to potential users and will provide sufficient computer power to perform the required computations at reasonable speed.  A printer is not required to use the program but is recommended for documentation of the results.  The software is being written in dBASE IV* (registered trademark of Ashton-Tate Inc.[3]).  Mention of trade names or commercial products does not constitute endorsement or recommendation for use by the US Army Corps of Engineers and will be distributed as a runtime or compiled program.  This frees the user from having to purchase dBASE IV* in order to run the DMF software.  The only additional software required is C or MS-DOS 3.0 or greater.

## THE PROGRAM

The DMF program is designed to assist users with implementing the DMF as easily as possible and guides the user through the DMF flow charts, asking for input where needed.  Once data have been entered into the computer's database, they are available whenever required throughout the entire application.  Although the DMF will be computerized as thoroughly as possible, there are still numerous places where decisions must be made by the user and results of these decisions entered into the computer program.  Online help screens are available throughout, explaining what information is requested at that particular point in the program and how to enter it.  After the opening screen, the user must decide which disposal option(s) he/she wishes to investigate (Figure 3).  All of the options may be pursued if desired.  One of the most time-saving facets of the DMF program is the ability to quickly investigate multiple disposal options allowing the user to determine which options are vi-able and of those which may be the most economically and environmentally sound.  Data that are unique to each disposal option must be entered before an option can be investigated.  The program is designed to inform the user what data will be needed in order to carry out the investigation of the disposal option selected. If the data are not available, the user may either postpone using the DMF until the data become available, or continue investigating other disposal options.

The next choice that the user must make is a determination concerning which sections of Federal law govern the proposed disposal; Section 103 of the Marine Protection, Research and Sanctuaries Act (MPRSA) or Section 404 of the Clean Water Act (CWA) (Figure 4).  After this determination has been made, the program checks to see if any of the conditions in the regulations are met that allow for immediate unrestricted disposal (Figure 5).  If these conditions are not met and section 404 of The CWA is the regulating law, then the user is asked to determine whether there is reason to believe that the material is contaminated based upon historical and currently available data (chemical spills in the area, industrial discharge, etc.).  If enough data are available to indicate that no likely contamination exists for this geographic area, then unrestricted disposal may be warranted.  If conditions exist that suggest contamination may be present, then the program moves to the next evaluation.

The next evaluation compares the test and reference sediments.  This requires that "bulk" chemical inventory data for both sediments be available.  If the data have been previously entered, then the user can simply select the data files to compare and continue with the program (Figure 6).  However, if new data must be entered, the user is prompted step by step to select the chemicals for which data are available and to enter that data.  The program saves the data in a file for future use and it may be edited at any time.  Options are also available for entering descriptive data about the sediment including site location, date sampled, date analyzed, etc.  These options are included so that whenever the data are recalled or printed, unique identifying information is always shown.

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[3]Mention of trade names or commercial products does not constitute endorsement or recommendation for use by the U.S. Army Corps of Engineers.

```
Your Disposal Options Are:

1. Aquatic Disposal
2. Upland Disposal
3. Consider Both Options

Please Choose Desired Option:
```

Figure 3. Representation of computer screen displaying possible disposal options to explore.

```
By Which Law is the Disposal Site Governed?

1. Section 404 of the Clean Water Act
2. Section 103 of Marine Protection, Research
   and Sanctuaries Act
3. Don't know, Need Help

Please Choose Desired Option:
```

Figure 4. Representation of computer screen showing menu choice of laws regulating disposal.

```
Best Description of the Proposed Dredged Material:

1.  Composed predominately of sand, gravel, rock or any other naturally occurring
    bottom material with particle sizes larger than silt, and the material is found in areas of
    high current or wave energy.

2.  Will be used for beach nourishment or restoration and is composed predominately of
    sand, gravel or shell with particle sizes compatible with material on the receiving
    beaches.

3.  The material is substantially the same as the substrate at the proposed disposal site
    AND the site from which the material proposed for dumping is to be taken is far
    removed from known existing and historical sources of pollution so as to provide
    reasonable assurance that such material has not been contaminated by such
    pollution.

4.  None of the above.

Please choose appropriate description:
```

Figure 5. Representation of computer screen showing description of disposal material and whether it meets the criteria for unrestricted disposal as per section 103 of the MPRSA.

```
┌─────────────────────────────────────────────────────────────────┐
│                                                                   │
│                         TIER I TESTING                            │
│                                                                   │
│                      Reference Sample Data                        │
│                                                                   │
│      1.  Enter a new set of reference sample data.                │
│      2.  Review or modify the chemical data in an existing sample data set. │
│                                                                   │
│                        Test Sample Data                           │
│                                                                   │
│      3.  Enter a new set of test sample data.                     │
│      4.  Review or modify the chemical data in an existing sample data set. │
│                                                                   │
│                        Comparison Testing                         │
│                                                                   │
│      5.  Perform comparison testing of reference and test sample data sets. │
│                                                                   │
│                    Please choose desired option:                  │
│                                                                   │
│                                                                   │
└─────────────────────────────────────────────────────────────────┘
```

Figure 6. Representation of computer screen showing selection of test and reference samples for Tier I comparisons.


After test and reference files have been selected (or created), they are compared against one another. All of the cases in which concentration of a chemical is higher in the test sediment than in the reference sediment are highlighted. The user must decide if the data indicate whether or not there is reason to believe contamination exists. (It is hoped that future work will allow portions of this to be computerized.) If there is no reason to believe that contamination exists, than unrestricted disposal may be investigated. If there is reason to believe contamination exists, then tests in successive tiers must be used to further evaluate the sediment.

The current version of the DMF program enters Tier II only if the user is interested in plant uptake (for upland disposal of fresh water sediments) or animal uptake (for aquatic disposal) of possible contaminants from dredged material. Other programs that have been written to deal with water quality impacts, mixing zone calculations, etc. are available (Schroeder 1988), but have not yet been merged with the DMF.

If the user wishes to pursue aquatic disposal, the DMF program will invoke the Bioaccumulation Program (BIOACC) to assist with the evaluation of benthic impacts by the proposed disposal. In Tier II the program calculates the Thermodynamically-defined Bioaccumulation Potential (TBP) (McFarland 1984; McFarland and Clarke 1986, 1987; and Clarke, McFarland and Dorkin 1988) for non-polar (neutral) organic chemicals. TBP allows the user to obtain estimates of potential chemical uptake from the sediments by aquatic organisms and compares these estimates with acceptable levels. The model uses data from the "bulk" chemical inventory and prompts for other needed data (i.e., organism lipid and sediment total organic carbon). If the user is unable to provide lipid data, the program uses default values from a database based on the scientific literature or estimates appropriate values from regression equations.

Because TBP is appropriate only for non-polar organic chemicals (e.g., DDT, PCBs, PAHs, dioxins, or dibenzofurans) if metals or polar chemical compounds such as organotins are a concern, laboratory testing in Tier III must be done. However, TBP may still be helpful in determining the potential bioaccumulation of the non-polar organics without resorting to expensive chemical analyses. TBPs for

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the test sediment data are compared to TBPs for the reference sediment chemicals and the results are presented to the user in the form of a chart showing which test concentrations exceed the reference concentrations.

If upland disposal for freshwater sediments is the option chosen then Tier II invokes the Plant Uptake Program (PUP) for estimation of heavy metal contamination. PUP was developed based upon a large volume of plant bioassay data (Folsom and Houck 1989). The user begins the estimation process by supplying data that were not included in initial sediment chemical analysis (i.e., sediment extractions with diethylenetriaminepentaacetic acid (DTPA), pH and organic matter). The data may be edited if necessary before the calculations are made. PUP will calculate the plant uptake estimations for all heavy metals for which data are available.

DTPA-sediment extraction comparisons are implemented as stated in paragraphs B47 to B49 of the DMF (Peddicord et al., 1986). Based upon the results of these comparisons, one of three conclusions may be reached: 1) no restrictions on upland disposal based upon metals contamination; 2) data is inconclusive leading to a local authority decision concerning disposal restrictions; or 3) an indication of possible metal contamination exists. This requires further evaluation of the test results and may necessitate a plant bioassay being performed.

Only in rare instances will Tier IV case-specific tests be required to allow a disposal decision to be made. Tier IV is designed to address the long-term effects of exposure to the dredge material. These tests should be designed to investigate specific issues that are potential cause for concern in this particular project. After all necessary information is gathered, disposal recommendations can be made. These may be unrestricted upland disposal, unrestricted aquatic disposal, restricted upland disposal or restricted aquatic disposal.

The DMF computerization effort is still undergoing development. While the concepts will remain basically the same, the implementation of these concepts will undoubtedly change as development proceeds. The DMF document is currently undergoing revision as is the Implementation Manual for ocean dumping. The software will be modified as needed to reflect current policy and laws for the disposal of dredged material.

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## APPENDIX I:  NOTES AND DEFINITIONS

Note A - Unrestricted ocean disposal at a designated site is acceptable if all other requirements of the regulations are satisfied.

Note B - Unrestricted ocean disposal is not acceptable.

DM -  Dredged material.

LPC -  It must be demonstrated that the limiting permissible concentration (LPC) would not be exceeded by existing data comparable to the data that would be generated if testing in Tier II, III, or IV, as appropriate, were conducted.

LC50 -  Acutely toxic concentration, i.e., lethal concentration to 50% of test organisms.

REF -  Reference material.

TBP -  Thermodynamically-defined Bioaccumulation Potential.

WQC -  Acute Water Quality Criteria.

# THE DEVELOPMENT OF A DECISION MAKING FRAMEWORK FOR THE EVALUATION OF SEDIMENTS CONTAINING 2,3,7,8-TCDD (DIOXIN)

by

Eric A. Stern[1] and John F. Tavolaro[2]

## INTRODUCTION

A major responsibility of the New York District (NYD) of the U.S. Army Corps of Engineers (COE) is to keep the Port of New York and New Jersey and its navigation channels open for safe navigation. The NYD has been dredging in the Port since 1888 and disposing of the dredged material offshore. Since 1914, dredged material has been disposed in the vicinity of the Mud Dump Site in the Atlantic Ocean, six miles east of Sandy Hook, New Jersey. This site was officially designated by the U.S. Environmental Protection Agency (USEPA) in 1984 as a dredged material disposal site. More than 90% of the 7-11 million cubic yards of sediment dredged annually from the Port of New York and New Jersey are disposed of at the Mud Dump Site. In the NYD, dredged material that is less than 90% sand and proposed for ocean disposal must satisfy the current USEPA/COE Ocean Dumping Testing Criteria (USEPA,COE 1977). The identification, bioassessment, water column/sediment interaction, and technical management of contaminants in dredged material is of major importance to the NYD's Federal and regulatory dredging programs. This paper will discuss the framework developed by the NYD to determine testing guidance values for dredged material contaminated with dioxin. Toxicologists, knowing the severe toxic effects of dioxin in experimental animals but being uncertain about comparable serious effects on people, call for more research. Environmental managers, who must make decisions based on this conflicting evidence, are left wondering what to do (Tschirley, 1986).

## DIOXIN IN THE PORT OF NEW YORK AND NEW JERSEY

It has been documented that certain faunal species within the lower Passaic River, Newark Bay and its tributaries as well as in the New York Bight apex are contaminated with 2,3,7,8-tetracholorodibenzo-p-dioxin (TCDD) (Belton, NJDEP, 1985). TCDD concentrations in species tested in the N.J. Department of Environmental Protection study exceeded the 25-50 part per trillion (pg/kg) Food and Drug Administration (FDA) levels of concern in tissue for fish caught by recreational fisherman in the Great Lake States. Exceptional low doses in laboratory test animals has shown a wide range of toxic responses in animals including carcinogenicity, teratogenicity, fetotoxicity, reproductive dysfunction and immunotoxicity (USEPA/NDS, 1987).

A major source of TCDD contamination in estuarine sediments is a former pesticide industrial site on the lower Passaic River in Newark, New Jersey. TCDD was formed at this plant as a by-product and co-contaminant in the production of 2,4,5-trichlorophenoxyacetic acid (2,4,5-T) during the years 1951-1969 (IT Corp, 1986). 2,4,5-T is also a constituent of Agent Orange, a defoliant used in Vietnam from 1962-1970. In 1960, an explosion at the plant of a 2,4,5-T containment vessel contaminated the upland soil. Plant outflows leading to the lower Passaic River as well as residual TCDD left over from the cleanup process might have also added TCDD to the river (IT Corp, 1986).

Concern was raised that TCDD contamination may have spread to adjacent waterways, contaminating the dredged material in Federal navigation channels and private berthing areas. Since dredged material from these waterways is normally disposed of in the ocean, it became evident that testing protocols and evaluative methods needed to be developed to determine whether dredged material suspected of being contaminated with TCDD was suitable for ocean disposal.

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## INTER-AGENCY DIOXIN COMMITTEE

In response to this effort, the NYD assembled a Federal Inter-Agency Dioxin Steering Committee in May 1986. This committee is comprised of representatives from USEPA (Region II), USEPA Research Laboratory (Narragansett, RI), U.S. Fish and Wildlife Service (USF&WS) (Absecon, NJ), USF&WS Fishery Contaminant Research Laboratory (Columbia, MO), NOAA/NMFS (Sandy Hook, NJ), USACOE Waterways Experiment Station (Vicksburg, MS) and the N.J. Department of Environmental Protection. It was the goal of the committee to develop decision guidance values for TCDD as well as to provide a basis for interpreting bioassay/bioaccumulation test results for dredged material containing TCDD.

The committee explored several approaches in evaluating TCDD concentration data. This data were obtained from Federal and private dredging projects where TCDD was detected. At first, the committee focused on an approach that utilized existing 10-day tissue bioaccumulation data rather than sediment data. This direction was proposed because of the unavailability of TCDD research to explain complex sediment-animal relationships in a marine system and because of analytical problems associated with TCDD testing in sediments. Previous studies measured the bioaccumulation of TCDD by organisms exposed to freshwater sediments (Isensee and Jones, 1975; Kuehl et. al., 1987). However, because of a lack of information on uptake and depuration rates as well as being unable to predict steady state concentrations, the committee expressed a lack of confidence in evaluating 10-day bioaccumulation tests for TCDD.

## LABORATORY INVESTIGATIONS

In October 1987, to address these concerns, the NYD began funding investigations of the long-term *bioaccumulation potential of TCDD and 2,3,7,8-tetrachlorodibenzofuran (TCDF) in contaminated* estuarine sediments. This research effort was performed using the resources of the USEPA Research Laboratory, Narragansett, RI. TCDF was also analyzed in the study since their physical, chemical and biological properties are similar to that of TCDD. Both TCDD and TCDF are extremely water insoluble and, like other hydrophobic compounds, have a strong affinity for particulate material associated with bottom sediments (Rubinstein, 1987). "Worst case" test sediment for this study came from the lower Passaic River where TCDD sediment concentrations were already documented (IT Corp., 1986). The objectives of study were as follows:

1. Empirically determine the long-term (180 days) bio-availability of TCDD and TCDF from known contaminated lower Passaic River sediment. TCDD bioaccumulation uptakes would be determined in three marine species: the sand worm, Nereis virens; the grass shrimp, Palaemontes pugio; and the hard clam, Macoma nasuta. The sandworm and the grass shrimp are organisms indigenous to the NY Bight and are currently used as the standard 10-day bioassay/bioaccumulation organisms in the evaluation of dredged material for ocean disposal in the NYD. Macoma was used in place of the NYD bioassay bivalve, Mercenaria mercenaria, since it demonstrates direct ingestion with the sediment.

2. Develop a relationship between concentrations resulting from 10-day exposures to steady state *concentrations.*

3. Determine the uptake and depuration kinetics for TCDD and TCDF and develop an empirical modelling approach for predicting steady state body burdens from sediment concentrations in the three marine species tested for this study.

A second study by the USEPA Research Laboratory was concurrently undertaken to determine the ambient TCDD concentration levels in sediment and tissues in the NY Bight apex. This study was necessary since Belton (NJDEP, 1985), reported TCDD tissue concentrations in commercially desirable fauna from the NY Bight, such as lobsters, exceeding the FDA action levels. The level of TCDD concentrations surrounding the Mud Dump Site will directly influence what criteria will be determined assuming a "no further degradation" approach.

## 2,3,7,8-TCDD CRITERIA DEVELOPMENT

While these two studies were underway, there was still a need to evaluate projects proposed for ocean disposal from areas known or suspected to be contaminated by TCDD. The committee was obliged to evaluate and interpret all available TCDD data for these projects without long-term verifiable bioaccumulation data. It was decided to use a conservative and accepted approach, using equilibrium partitioning (McFarland, 1984) to estimate the maximum bioaccumulation tissue concentration from TCDD bulk sediment data. Besides bulk sediment data, this approach takes into account the total organic carbon (TOC) content of the project sediment and percent body lipid of an individual target organism.

This approach has been used for other organohalogens, such as for PCB contaminated sediment (McFarland and Clarke, 1986) but has not been verified for TCDD. Since a few highly chlorinated PCB congeners and dioxins are somewhat similar in their sediment/biota interactions, the committee adapted this approach to making hypothetical tissue TCDD predictions from bulk sediment concentrations. This predictive modelling approach assumes the most conservative "worst case" conditions. These conditions are that (1) TCDD would be in a biologically available state; (2) TCDD would be physically available to the organism; and (3) the organism would receive a lifetime exposure to TCDD.

After the committee was assembled, testing laboratories developed and refined their TCDD analytical techniques to achieve lower detection limits to the part per trillion detection limit in sediment and tissue matrixes. Prior to 1986, there was concern that commercial state-of-the-art detection limits for TCDD at one part per billion were too high since concentrations on the order of parts per quadrillion had lethal effects on some laboratory animals. With greater confidence in achieving the lower detection limits, TCDD bulk sediment data is now tested for both private and Federal dredging projects. To evaluate this data, the NYD developed an interim three tiered testing approach to be used while the long-term bioaccumulation study was in progress (Figure 1). It was also thought that this approach would be consistent with the tiered approach currently being proposed by the Corps and EPA in their revision of the Ocean Disposal Testing Criteria.

### PROPOSED 3-TIER INTERIM 2,3,7,8-TCDD TESTING APPROACH

Tier 1:

Bulk sediment analysis would be the first tier. Because of TCDD's hydrophobic nature and its strong affinity for silts and clays, extra sample sites should be considered than usual to adequately characterize the project area. Representative cores would be taken to project depth, individually homogenized and analyzed separately for TCDD at a detection limit of one part per trillion. TOC concentrations are also analyzed for each core. If TCDD is not detected in any of the cores, obviously no further TCDD analysis would be required. If any measurable concentrations of TCDD are reported, the evaluation proceeds to the second tier.

Tier 2:

In this tier, TCDD bulk sediment and TOC concentrations results from the first tier would be used in conjunction with the thermodynamic method of predicting the maximum tissue bioaccumulation potential. Representative lipid percentages of 1%, 3% and 6% are used in the calculation instead of one lipid value for a target species. This lipid range is characteristic of NY Bight species that could come into contact with the sediment. These values also encompass the lipid range of the bioassay/bioaccumulation testing species presently being used by the NYD.

The NYD proposed using evaluative criteria based on FDA tissue levels of concern to interpret TCDD projected tissue levels.
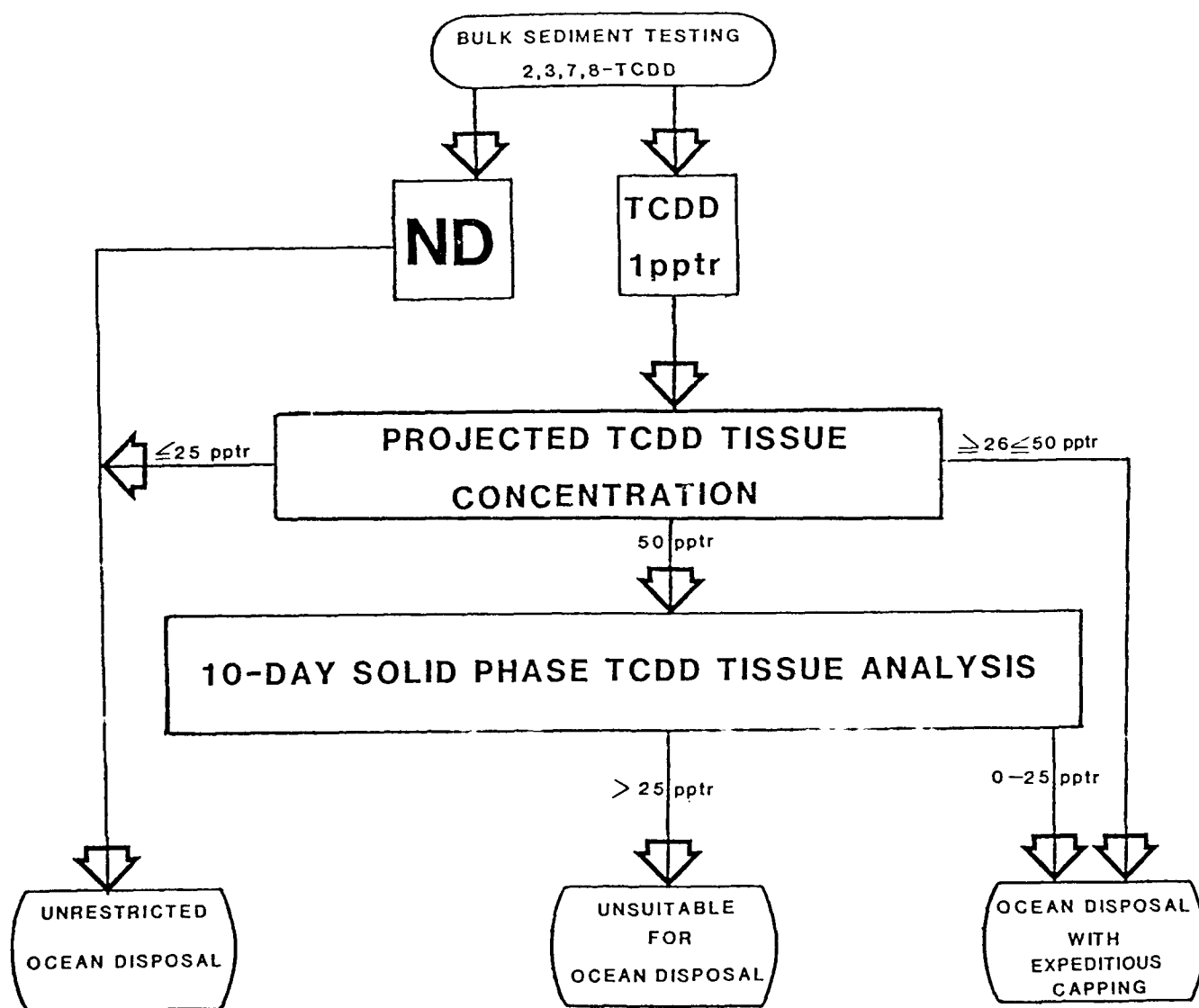
Figure 1. Interim Three Tier 2, 3, 7, 8-TCDD Testing Approach

The FDA criteria were as follows:

1. For levels in fish below 25 pg/g, little cause for concern;
2. For levels in fish between 25 and 50 pg/g, restriction of intake to no more than one meal of fish per week;
3. For levels exceeding 50 pg/g, the State should consider a ban on the consumption of fish from these areas.

The NYD transposed the FDA criteria to an ocean disposal management framework for projected TCDD tissue concentrations.

The transposed criteria were as follows:

1. For all calculated tissue levels at 25 pg/g or below, unrestricted ocean disposal will be allowed;
2. For all calculated tissue levels between 26 and 50 pg/g, only ocean disposal with expeditious capping (within 10 days) will be allowed;
3. For all calculated tissue levels above 50 pg/g, other alternatives to ocean disposal must be considered, or proceed to Tier 3.

If the project or any portion of a project failed the first two tiers, there is the potential that the dredged material may cause bioaccumulation of TCDD above 50 pg/g in the long term. But we must recognize that the kinetic uptake of TCDD is extremely slow and that this predictive modelling approach uses an extremely conservative evaluative criteria in Tier 2, assuming that the TCDD is bioavailable and that the organism will be exposed to TCDD for life. It was therefore important to determine whether the potential bioaccumulation levels revealed by the Tier 2 analysis were realized by actual sediment animal exposures. The research presently being performed by USEPA Research Laboratory will give us some answers to this question in detail, but in the interim, a 10-day bioaccumulation test could still be of value. The 10-day bioaccumulation test tells us the actual levels of bioaccumulation realized after 10 days. This gives indication of the time span within which capping should occur in order to cap and therefore isolate the dredged material before organisms have had the chance to bioaccumulate unacceptable levels of TCDD.

Tier 3:

In the third tier, project sediment would be composited and a 10-day solid phase bioassay/bioaccumulation analysis for TCDD be performed on the sandworm, Nereis virens. Five replicates at a detection limit of one part per trillion would be performed. The sandworm was used for this analysis since it has a greater lipid content (7-8%) as compared with the clam (1-2%). The sandworm's infaunal nature suggests that it could serve as a target organism in which we would most likely be able to detect measurable concentrations of TCDD.

For Tier 3, the NYD proposed to use the lowest FDA level of concern, of 25 pg/g and below, for the cutoff for ocean disposal with expeditious capping. An average tissue concentration of less than 25 pg/g in Tier 3 would be determined suitable for ocean disposal with expeditious (within 10 days). Any averaged tissue concentration greater than 25 pg/g would be unsuitable for ocean disposal at the present time.

The concept of this tiered approach was, for the most part, accepted among the committee. However, data generated from the bulk sediment, predictive modelling and 10-day solid phase bioassay/bioaccumulation analyses were difficult to translate to the FDA criteria. Bulk sediment concentrations of 35 pg/g usually exceeded the FDA criteria when its projected long-term tissue bioaccumulation potential was calculated in Tier 2. Tier 3 would most always be approached since the predictive model is extremely conservative.

It was also observed that after the 10 day-bioassay/bioaccumulation tissue analysis for the sandworm and clam, even with 2,3,7,8-TCDD bulk sediment concentrations above 250 pg/g, tissue concentrations never exceeded 10 pg/g in Tier 3 . Therefore, NYD "worst case" bulk sediment concentrations would always pass the lowest FDA criteria of 25 pg/g for ocean disposal in Tier 3. It was apparent that 10 days was insufficient time to observe uptake as well as needing to know how many days were required for tissue concentrations to reach steady state. The committee decided to wait for the USEPA Narragansett studies to be completed before making any decisions on an interim criteria.

## PRELIMINARY RESULTS

Long-term bioaccumulation and sediment chemistry results has been submitted to the NYD by the USEPA Narragansett Research Laboratory. The mean TCDD bulk sediment concentration for the lower Passaic River test sediment was $656+97$ pg/g and $334+6$ pg/g for TCDF. For TCDD, the sandworm reached steady state approximately 120 days after exposure to the test sediment, and finally reaching a maximum tissue concentration of $422+103$ pg/g at 180 days. Sandworms exposed to test sediment for 70 days were held in control sediment for 120 days and allowed to depurate. The sandworm depurated to approximately one-half its maximum concentration at 180 days. Sandworms did not accumulate high concentrations of TCDF. Bioaccumulation levels increased slowly to steady state at 42 days and then maintained this level throughout the 180 day exposure. After 180 days, the TCDF concentration was $112+51$ pg/g, approximately one-fourth of the maximum TCDD concentration for the sandworm.
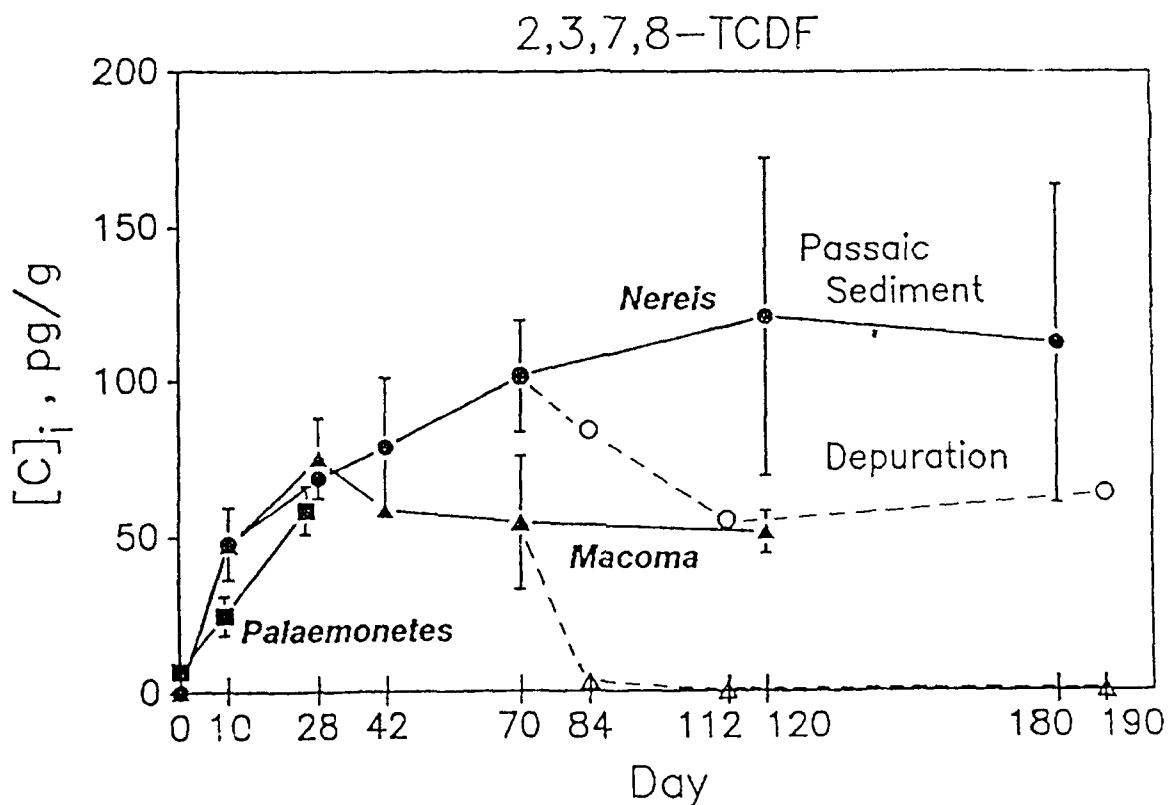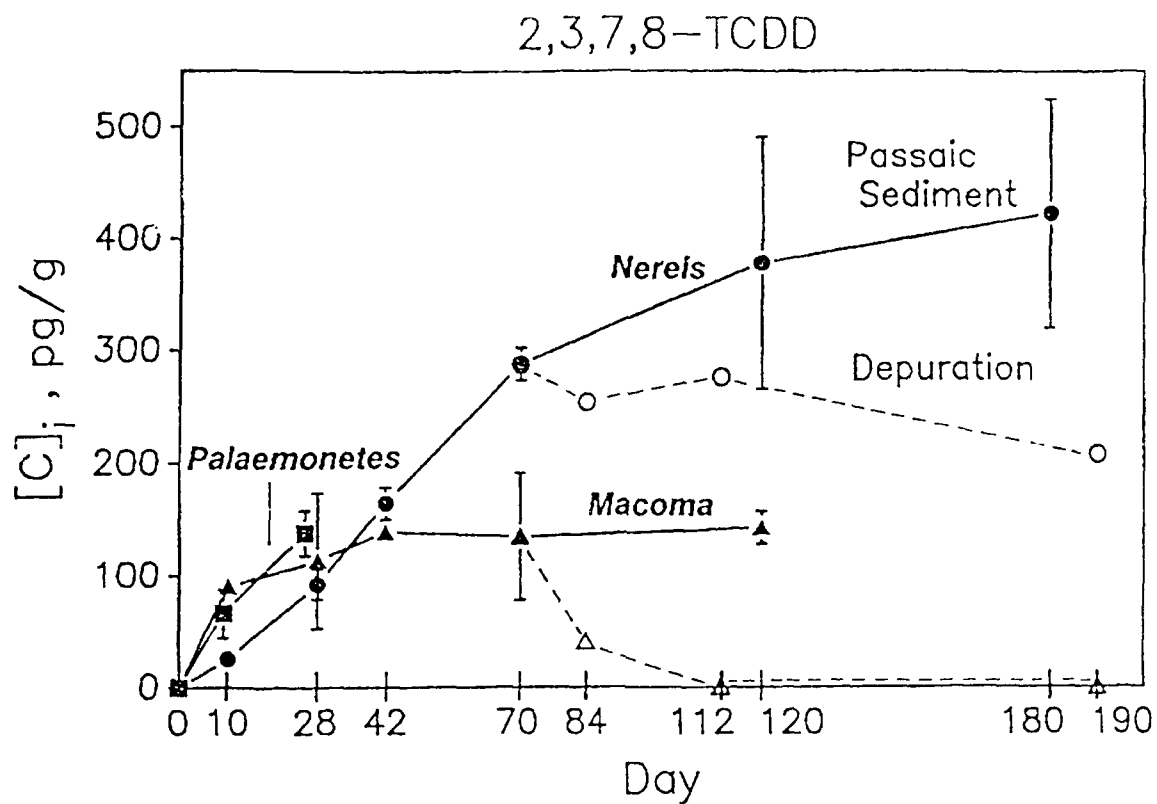
The clam rapidly bioaccumulated and depurated TCDD and TCDF, attaining steady state in approximately 40 days. Maximum tissue concentration at 120 days were $142+20$ pg/g for TCDD and $51.4+6.8$ pg/g for TCDF. The clam rapidly depurated from 70 days to near background levels. The grass shrimp after 28 days, accumulated $138+20$ pg/g for TCDD and $58.8+7.7$ pg/g for TCDF (Figure 2) (Rubinstein et al. 1989).

## CONCLUSIONS

1.  The 3-tiered decision framework approach proposed by the NYD for evaluating sediments contaminated with TCDD would be valid if a 28-day bioassay/bioaccumulation test is considered in Tier 3. A 10-day solid phase bioassay/bioaccumulation test does not address the realized bioaccumulation potential of TCDD and TCDF.

2.  Preliminary results from the USEPA Narragansett Research Laboratory show that TCDD, like PCB's, achieves steady state but takes longer. Therefore, bioassay/bioaccumulation testing for TCDD would be valid for ecological predictions.

3.  Since laboratory depuration is rapid after the organisms exposure to TCDD is cutoff, strategic capping within 10 days would be an effective tool for managing these types of sediments.

4.  Testing guidance values for TCDD and TCDF still need to be evaluated by the Federal Inter-Agency Dioxin Committee to determine acceptable calculated concentrations in Tier 2.

## ACKNOWLEDGMENTS

Figure 2. Passaic River Test Sediment Accumulation Curves

(Rubinstein et al. 1989)

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# A LOW-COST COMBINED CHEMICAL/BIOLOGICAL PROCEDURE FOR THE ANALYSIS OF CONTAMINATED SEDIMENTS

by

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## INTRODUCTION

Both aquatic sediments and terrestrial soils can act as sinks for contaminants. Conversely, these sediments and soils can later act as sources for the same contaminants when perturbations such as construction, site remediation, tidal pumping, benthic feeding, storms, floods, or dredging and disposal occur. Clearly, the identification of biologically active contaminants is necessary to define risks, both to human health and the environment. This identification process is complicated by the variability in biological activity and chemical composition of the contaminants in the sediments or soils in question.

Whenever it is necessary to evaluate dredged sediments for the presence of chemical contamination using chemical analysis and/or bioassay, substantial costs are involved. Biologically active compounds have been identified from sediments by using high performance liquid chromatography (HPLC), and gas chromatography/mass spectrometry (GC/MS). These high-resolution methods are costly and time-consuming, considering the high degree of sample variability, the vanishingly small amount of the contaminant analyte generally present, and the large numbers of analyses required. Frequently, compromises must be made between the number of sediment samples needed for adequate characterization of an area, and the cost of analyzing all the samples. The high resolution chemical analyses that are so costly and the suite of expensive bioassays that are typically done are not always necessary during the initial stages of a dredging and disposal regulatory evaluation. When chemical contamination of sediments in concentrations sufficient to warrant concern is suspected, lower resolution analytical procedures may be capable of characterizing the problem more cost-effectively than expensive HPLC or GC/MS methods.

Assays of mutagenesis exist (Holstein, et al., 1979) and generally, there is a high correlation between in vitro mutagenicity and corresponding in vivo carcinogenicity (McCann, et al., 1975). Indeed, the ubiquitous nature of DNA supports extrapolation of mutagenicity results between species (Hoffman, 1982). By far, the most widely used and extensively validated, short-term, mutagenicity test is the Ames Salmonella/mammalian-microsome mutagenicity test (Ames, et al., 1975). It has high sensitivity (80-90% of carcinogens are detected), is simple, rapid, and inexpensive, and therefore has gained widespread acceptance as an initial step in screening environmental contaminants (McCann, et al., 1975; Maron, et al., 1983; and Butler, 1984).

The Ames assay can detect both types of point mutations (base-pair substitution and frame-shift mutation) by choice of the proper Salmonella typhimurium tester strain. The assay involves placing test compound and mutant bacteria on minimal agar plates (both with and without metabolic activators called S-9). A trace amount of histidine allows some bacterial growth, but only bacteria that have mutated back to the "wild-type" can form colonies in the absence of histidine, which is a vital amino acid that the mutant strains require. These "revertant" colonies can be enumerated to determine the mutagenicity of the compound in question.

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The *Ames Salmonella*/mammalian-microsome mutagenicity test is particularly well-suited to testing complex environmental mixtures and has been used extensively to evaluate airborne contaminants (Butler, 1984). Bjorseth, et al. (1987) and Butler (1984) used high-performance thin-layer chromatography (HPTLC) separations of complex mixtures of airborne environmental contaminants prior to Ames assay to separate groups of mutagens.

We have developed, and are evaluating, an integrated chemical separation/analysis/bioassay for contaminated sediments. The method utilizes the inexpensive, rapid, and sensitive methodology of HPTLC, and the *Ames Salmonella*/mammalian-microsome mutagenicity test (Ames, et al., 1975). This HPTLC/Modified Ames Test is a procedure that identifies and quantifies chemicals by classes and provides an indication of bioactivity through microbial mutagenicity and growth inhibition or stimulation. HPTLC is used to separate chemicals by classes according to polarity using sequential elution on standard and reversed-phase thin-layer chromatographic plates. Following the development of the plates and identification of the compound classes present, quantitation can be accomplished using a scanning densitometer. Bioassays performed directly on duplicate developed HPTLC plates by combining bacteria with soft agar and pouring that over the HPTLC plates can then be used to give an indication of the mutagenicity of the sample.

## MATERIALS AND METHODS

Soil samples were obtained from a terrestrial site (a U.S. Environmental Protection Agency - EPA designated superfund site in New Jersey), and sediments from a freshwater aquatic site (Sheboygan Harbor Wisconsin). Additionally, organic extracts of marine aquatic sediments were obtained from several sites with varying degrees of contamination in Puget Sound, Washington (Elliot Bay, Eagle Harbor, Commencement Bay) through the EPA Comparison of Bioassays for Assessing Sediment Toxicity in Puget Sound, part of the Puget Sound Estuary Program (PSEP).

The organic sediment extracts from PSEP had been prepared by the method of MacLeod, et al. (1985). The sediments from New Jersey and Wisconsin were extracted using the method of Reilly, et al. (1986). The extracts were used to spot individual lanes of pre-cleaned, scored HPTLC plates from a variety of vendors including: Whatmann LHP-K, with a preadsorbant strip; EM Science Silica Gel 60 F254; and Analtech HP-HL and HP-RPS. Repeated trials demonstrated that HPTLC has the ability to separate the components of sediment extracts with concentrations up to 10mg/ml of organic matter provided a volume of no more than 25ml of extract was used to make the spot (more volume caused the spot to become too large to chromatograph properly). This required 100ml aliquots of each extract to be reduced to near-dryness under dry N2 and reconstituted to 25ml with the same extract solvent. After concentrating this way, spotting was performed near the center of each lane of a HPTLC plate.

These spotted plates were then developed as described in Reilly, et al. (1986) using the solvent development systems listed in Table 1. Separation of samples for the purpose of identifying individual Polycyclic Aromatic Hydrocarbons (PAH) was accomplished by using reversed-phase HPTLC plates (plates having a non-polar solid-phase) and mobile-phase consisting a mixture of methanol:dichloromethane: water in the ratio of 20:3:3.

Standards representative of each chemical class were pooled to make a combined standard and chromatographed on each plate along with samples and blanks. A separate plate was prepared for the determination of each class of compounds and the entire plate was treated to make these compounds visible. The treatment required, as well as the proper development system, are given in Table 1. Table 2 gives the standards used for each class as well as their manufacturer, purity, relative migration (Rf), and appearance both before and after visualization treatment.

In every case, both combined standard and individual compounds were tested to identify their appearance after visualization by spotting a piece of Bakerflex plastic coated TLC plates, and treating it with the appropriate visualization system. This was done prior to observing the HPTLC plates so that a positive identification could be made, based on color or fluorescence. The advantage of doing this initial test is that the location of the standards are known because no chromatography has been done,

Table 1. Methods Used for Chemical Class Detection
in HPTLC/Ames Assay.

| Chemical Class | Development System* | Visualization System | Reference Method |
|---|---|---|---|
| PAH | A | UV Light (254nm) | 10 |
| Nitro-PAH | A | Iodoplatinate Reagent | 11 |
| Carbonyls | B | 2,4-dintrophenyl hydrazine; KOH | 12 |
| Phenols | B | Ferric Chloride-HC1 Heat | 13 |
| Carboxylic Acids | B;C | Bromcresol Green | 6 |
| Aza-arenes | C | Potassium Iodoplatinate Reagent | 5 |
| Halogenated Hydrocarbons | A,B | Fluorescein; Rhodamine B; KOH | 14 |

* Development Systems: A=Toluene/Hexanes (5:1); B-Dichloromethane (100%); C=Acetone (100%)

ensuring proper identification of the resultant color as the standard. Additionally, the Bakerflex plates are relatively inexpensive. The relative mobility of the individual compounds, with respect to the solvent front (Rf) or with respect to the migration of a known standard (Rx), could then be made by comparing the bands on the HPTLC plate to the spots made on the Bakerflex plates.

The visualization procedures used are referenced in Table 1 and do not differ appreciably from the methods described by Butler (1984) with the exception of the methods used to visualize halogenated hydrocarbons (EM Laboratories Inc., 1976) and to positively identify the specific PAH present at a given Rf.

PAH were identified using a scanning densitometer to selectively irradiate specific spots of the HPTLC plate with five different wavelengths of UV light and measure the fluorescence of the spots through four different cut-off filters. The resultant reflectance data were correlated and plotted using a three-dimensional computer program. This produced a specific, three-dimensional, "fingerprint" that was characteristic for each PAH compound, and useful for compound identification.

The methods used to assay the mutagenicity of whole sediment extracts are those of Ames, et al. (1975). Standard Ames procedures were applied to a dilution series of each of the PSEP extracts in order to determine if there was a correlation between exposure concentration and mutagenicity (number of reversions) normalized for background levels of mutations (mutagenic activity ratio). HPTLC separations of the whole sediment extracts were assayed to determine the mutagenicity of the individual classes of compounds present using the methods of Bjorseth, et al. (1982), Butler (1984) and Reilly, et al. (1986).

## RESULTS

The results of HPTLC analysis of combined standards are given in Table 2. Shown in that table are the relative mobilities of the individual compounds chromatographed, as well as color changes or fluorescence information used in demonstrating their identity. It should be noted that different plate coating materials (solid-phase) can affect not only the Rf value, but also the perceived colors of the compounds. This latter fact can be demonstrated by the yellow color of toluene:hexanes (5:1) development of Aroclor 1254 observed on EM Science Silica Gel 60 F254 plates, and the inability to visualize Aroclor 1254 on the Whatmann LHP-K plates prior to visualization treatment.

The solvent system can affect both the Rf and the color detected upon visualization. Most carboxylic acid class members had different Rf values when developed in 100% dichloromethane than when developed in 100% acetone. Color differences are evident between the toluene:hexanes (5:1) and dichloromethane developing systems for Aroclor 1254 both on the EM Science Silica Gel 60 F254 and on the LHP-K plates.

Many of the compounds tested could not be separated by the developing system chosen. That is, they have the same or overlapping Rf values. In most cases, of overlap the compounds of the class traveled with the solvent front, or remained near the origin. Therefore, it is feasible that these compounds could be separated by using a specific solvent system. In fact, this is what was done to separate individual PAH, as shown with their relative migrations in Figure 1. However, identification of chemical classes should be adequate for screening purposes. Higher resolution techniques (HPLC, GC/MS) could be invoked, if needed, to identify specific compounds after one of the chemical classes was shown to be present in a given sample.

Blank chromatograms were largely devoid of any positive test results, though a few classes (carboxylic acids and aza-arenes) tested slightly positive at the solvent front. In as much as this procedure concentrates the solvents and their contaminants by 2000 times compared to the solvents at the start of the procedure, this is not surprising.

The results of the HPTLC chemical class screening of the environmental samples are summarized in Table 3. All of the classes of compounds tested using the methods given in Table 1 were shown to be present in both sets of sediment samples, and in some of the organic extracts obtained from PSEP. It should be stressed that classes of compounds will be observed on the chromatographic plates. Observation of a positive image does not confirm the presence of a specific compound, only the presence of one or more members of the chemical class. Figure 2 demonstrates this. It is a diagram of the 100% dichloromethane development of a chromatographic plate (EM Science Silica Gel 60 F254) after being treated to visualize halogenated hydrocarbons. Lane one contained the dichloromethane extract of terrestrial sediment. Lane two contained the standard (Aroclor 1254, 1mg/ml). Lane three was spotted with a dichloromethane extract of the Sheboygan Harbor sediments, while lane four contained the blank extract. Regions having halogenated hydrocarbons are visualized as dark or dark red spots that fluoresce slightly under UV radiation. They are shown in Figure 2 as dark spots.

The Aroclor 1254 standard (Figure 2, lane 2) all migrated with the solvent front (Rf = 1). Both terrestrial (Figure 2, lane 1) and aquatic (Figure 2, lane 3) sediment extracts exhibited the same band (Rf = 1), indicative of halogenated hydrocarbons, though not necessarily indicative of Aroclor. Although compounds other than halogenated hydrocarbons could possibly be present in the band at the solvent front, they would not be colored by the specific reagents used. The terrestrial soil extract (Figure 2, lane 1) also had a broad, slowly migrating band, and a smaller moderately migrating band. These stained areas on the chromatographic plate indicate the presence of halogenated hydrocarbons that clearly are not Aroclor, as evidenced by their different pattern of migration. Characterization of chemical compounds is possible by both color during visualization and migration distance.

The other chemical classes also tested positive for the presence of class members other than those standards chosen. If required, the identity of the unknown class members can be determined by refining the separation technique used, expanding the number of class-member standards co-

Table 2. Mobilities (Rf) and Color Changes of Standard Compounds Using Specified Development and Visualization Systems.

| Standards by Class | Manufacturer[1] | Purity % | Rf | Visible or (Fluorescence) Color | |
|---|---|---|---|---|---|
| | | | | Untreated | Treated |
| **PAH** | | | | | |
| anthracene | A | 99.9+ | 1 | white | (turquoise) |
| fluoranthrene | A | 98 | .92-.98 | white | (blue) |
| benzo(ghi)perylene | A | Rgt. | .90-.99 | white | (turquoise) |
| **NITRO-PAH** | | | | | |
| o-nitrobenzaldehyde | A | 98 | ND | white | red |
| 2,4-dinitrophenyl hydrazine-HC1 | A | Rgt. | .34-.41 | yellow | yellow |
| 9-nitroanthracene | A | 97 | .82-.88 | yellow | yellow |
| 1,5-dinitro napthalene | A | Rgt. | .3 | white | white |
| **AROMATIC CARBONYLS** | | | | | |
| 1-pyrenecarbos aldehyde | A | 98+ | .96-1.0 | (yellow) | orange |
| 9-acetylphenanthrene | A | >97 | .96 | ND | yellow |
| **KETONES** | | | | | |
| xanthone | A | 99 | 1 | ND | yellow |
| anthrone | A | >97 | 1 | (aqua) | yellow/(blue) |
| **QUINONES** | | | | | |
| anthraquinone | F | $\approx$99 | .14-.18 | (pink) | yellow/(blue) |
| benz(a)anthracene-7,12-dione | | 99 | 1 | yellow | yellow |
| **CARBOXYLIC ACIDS** | | | | | |
| succinic | A | 99+ | $.02\text{-}.1^2$ | (white) | yellow |
| | | | $.02\text{-}.1^3$ | (white) | yellow |
| 2-napthanoic | A | 99+ | $0\text{-}.06^2$ | (blue) | yellow |
| | | | $.02\text{-}.1^3$ | (blue) | yellow |
| 9-anthrene | P&B | Rgt. | $0\text{-}.06^2$ | (blue) | yellow/(blue) |
| | | | $.02\text{-}.1^3$ | (blue) | yellow/(blue) |
| 1,2-napthanoic dicarboxylic acid | P&B | Rgt. | $.5\text{-},6^2$ | (blue) | yellow/(blue) |
| | | | $.96\text{-}.98^3$ | (blue) | yellow/(blue) |
| stearic | F | Rgt. | $.06\text{-}1.2^2$ | ND | yellow/(orange) |
| | | | $.3\text{-}.5^3$ | ND | yellow/(orange) |

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| Standards by Class | Manufacturer[1] | Visible or (Fluorescence) Purity % | Rf | Color Untreated | Treated |
|---|---|---|---|---|---|
| **AZA-ARENES** | | | | | |
| acridine | A | >97 | .9 | yellow/(blue) | black/(green) |
| 5.6-benzoquinoline | A | 99+ | 1 | (blue) | black/(red) |
| carbazol | A | 99+ | 1 | (blue) | black/(purple) |
| **PHENOLS** | | | | | |
| 2-hydroxycarbazole | A | 99 | .12-.2 | ND/(purple) | blue |
| α-napthol | A | 99 | .50-.58 | (purple) | orange |
| 9-hydroxyfluorene | P&B | min.95 | ND | white | ND |
| **HALOGENATED HYDROCARBONS** | | | | | |
| Aroclor 1254 | M | | $1^{4,5}$ | yellow | (red) |
| | | | $1^{2,5}$ | dark | (red) |
| | | | $1^{4,6}$ | ND | (red) |
| | | | $1^{4,6}$ | (purple) | (red) |

1. Manufacturers: A = Aldrich; F = Fisher; P&B = Phaltz & Baum; M = Monsanto.

2. Developed with Dichloromethane (100%).

3. Developed with Acetone (100%).

4. Developed with Toluene/Hexanes (5:1).

5. Using EM Science Silica Gel 60 F254 HPTLC Plates.

6. Using Whatman LHP-K Plates.

chromatographed until trial and error reveals the class member present in the sediment extract, or, by referring to the literature to find likely class members with a migration behavior similar to the unknown in the sediment extract.

The integrated assay was tried on prepared extracts obtained as a part of the PSEP program. Simple Ames tests performed on the whole extract (no chromatography prior to testing) showed a clear relationship between extract concentration and the number of mutations observed or mutagenic response ratio (Figure 3). This response held at concentrations above 100% to around 500% of the initial concentration, using Ames Salmonella tester strain TA-100 for extracts obtained from Commencement Bay (Figure 3a), and was observed for much lower doses ranging from 0.1% to 1.0% of the original extract concentration using tester strain TA-98 on sediment extracts from Eagle Harbor (Figure 3b). Use of the combined Ames/Modified HPTLC assay with reversed phase HPTLC plates on the organic extracts obtained from Commencement Bay identified a band with Rf = 0.61, and Rx (relative to Benzo-a-pyrene) = 1.469, as clearly mutagenic.

# IDENTIFICATION OF PAH'S
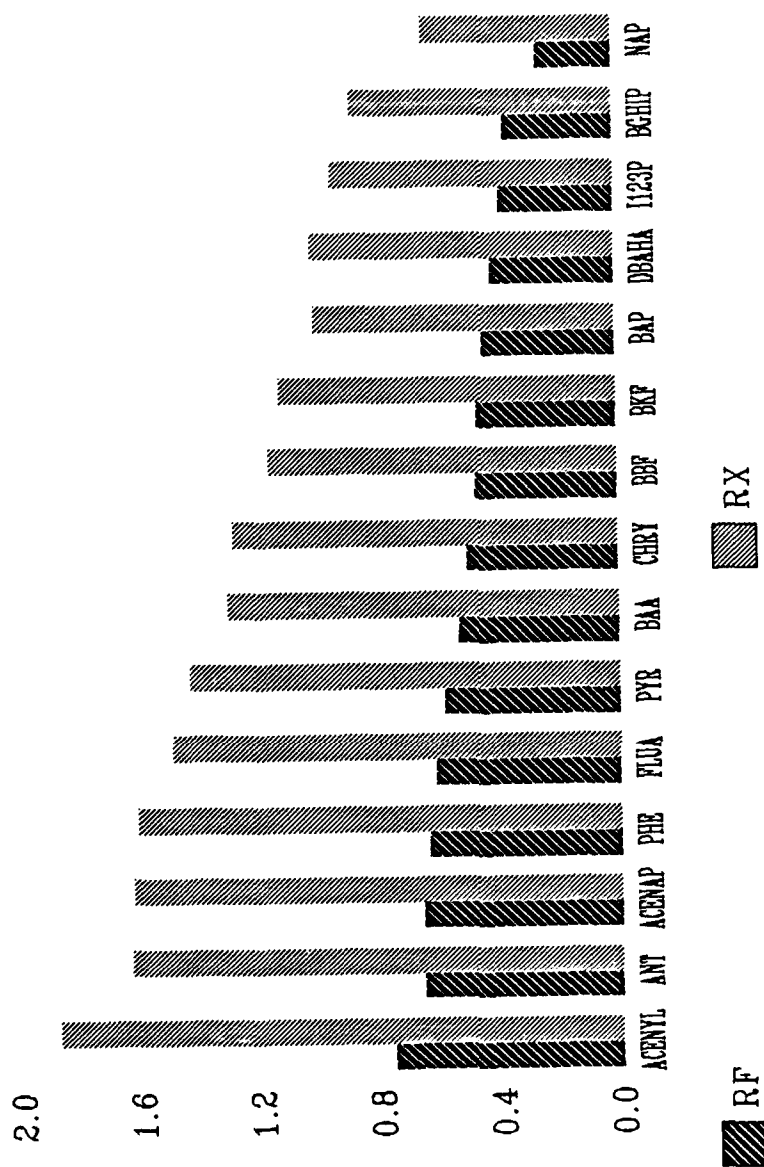
## USING RELATIVE MOBILITIES



**FIGURE 1**

IDENTIFICATION OF PAH'S USING RELATIVE MOBILITIES. Rf is the relative mobility with respect to the solvent front. Rf = distance from the origin to the center of the compound divided by the distance from the origin to the solvent front. Rx is the relative mobility with respect to a co-chromatographed standard, in this case Benzo (a) pyrene. Rx = distance from the origin to the center of the compound divided by the distance from the origin to the center of the spot of Benzo (a) pyrene. The Rf and Rx are given for acenapthylene (ACENYL), anthracene (ANT), acenapthene (ACENAP), phenanthrene (PHE), fluoranthene (FLU), fluoranthene (PYR), benzo (a) anthracene (BAA), chrysene (CHRY), benzo (b) fluoranthene (BBF), benzo (k) fluoranthene (BKF), benzo (a) pyrene (BAP), dibenzo (a,h) anthracene (DBAHA), indeno 1,2,3 pyrene (I123P), benzo (g,h,i) perylene (BGHIP), and naphthylene (NAP).

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Table 3. Chemical Classes Detected in Samples.

| Chemical Class | Terrestrial Samples | Aquatic Samples | Blank | Elliot Bay | Eagle Harbor | Commence-ment Bay |
|---|---|---|---|---|---|---|
| PAH | P* | P | ND* | P | P | P |
| Nitro-PAH | P | P | ND | NT* | NT | NT |
| Aromatic Carbonyls | P | P | ND | P | P | P |
| Ketones | P | P | ND | P | P | P |
| Quinones | P | P | ND | NT | NT | NT |
| Carboxylic Acids | | | | | | |
| DCM extracts | P | P | ND | NT | NT | NT |
| ACE extracts | P | P | D* | NT | NT | NT |
| Aza-Arenes | P | P | ND | NT | NT | NT |
| Phenols | P | P | ND | P | P | P |
| Halogenated Hydrocarbons | | | | P | P | P |
| on fluorescent plates | | | | | | |
| CX extracts | P | P | ND | NT | NT | NT |
| DMC extracts | P | P | ND | NT | NT | NT |
| on standard plates | | | | | | |
| CX extracts | P | P | ND | NT | NT | NT |
| DCM extracts | P | P | ND | NT | NT | NT |

* P=Present; ND=Non-detectable; D=Detectable; NT=Not tested

Figure 1 is a histogram showing the Rf and Rx of the most commonly occurring PAH. The Rx of the unknown mutagen is similar to the Rx for pyrene (Rx = 1.43) and fluoranthene (Rx = 1.49), indicating that the unknown may be one these two PAH. To resolve the question, the UV "fingerprints" of the three compounds were compared. The UV "fingerprint" of the unknown (Figure 4a) shows similarity to the UV "fingerprint" for fluoranthene (Figure 4b), indicating that the unknown is indeed fluoranthene, a known and common mutagen. Pyrene, on the other hand, is not a mutagen, and therefore would not have caused mutation of the tester bacteria. Bulk sediment analyses of the same sediment sample, performed by a participant of the PSEP program, subsequently identified the sediment as containing fluoranthene.

## CONCLUSIONS

The results disclosed in this paper clearly demonstrate that HPLC techniques can separate mixtures of environmental contaminants and allow visualization of compounds by chemical class. PAH, nitro-PAH, carbonyls, phenols, carboxylic acids, aza-arenes, and halogenated hydrocarbons can be identified by the use of the methods summarized here. Other classes of chemicals not discussed in this study may also be identifiable using different visualization systems. The HPTLC/Modified Ames Test

appears to be effective for all organic chemical contaminants, with the exception of the super-potent toxicants such as dioxin, which are biologically effective at levels below the chemical detection limit of HPTLC. Metal contaminants are not amenable to the procedure, because they are difficult to separate chromatographically. The extraction steps used in this HPTLC methodology are the same as those used for other high-resolution chromatographic analyses. Therefore, HPTLC does not preclude subsequent high-resolution analyses. However, the cost per sample for this combined chemical/biological sediment assessment procedure is a small fraction of the cost of higher resolution procedures currently in use.

An additional advantage is that the procedure provides an assessment of sediment genotoxicity, a feature that is lacking in all but a few very specialized bioassays. HPTLC plates developed as described in this study, but not treated with any of the previously mentioned visualization systems, can be incubated with media and appropriate Ames tester strains. If colonies develop, a mutation has occurred (and therefore the presence of a mutagen). The extract can be re-chromatographed. These second plates can be exposed to the visualization systems sequentially until the chemical class of the mutagenic compound can be identified as it occurs at the same Rf as the bacterial colony formation.

There are several advantages to using Ames testing of sediment extracts chromatographed on HPTLC plates as a screening procedure for environmental contaminants. Many samples can he run simultaneously, reducing cost. Samples not showing mutagenic activity can either be eliminated from further testing (thus further reducing cost, and increasing throughput of other samples), or given a lower analytical priority (thus speeding the analytical procedure, and directing further analytical effort).
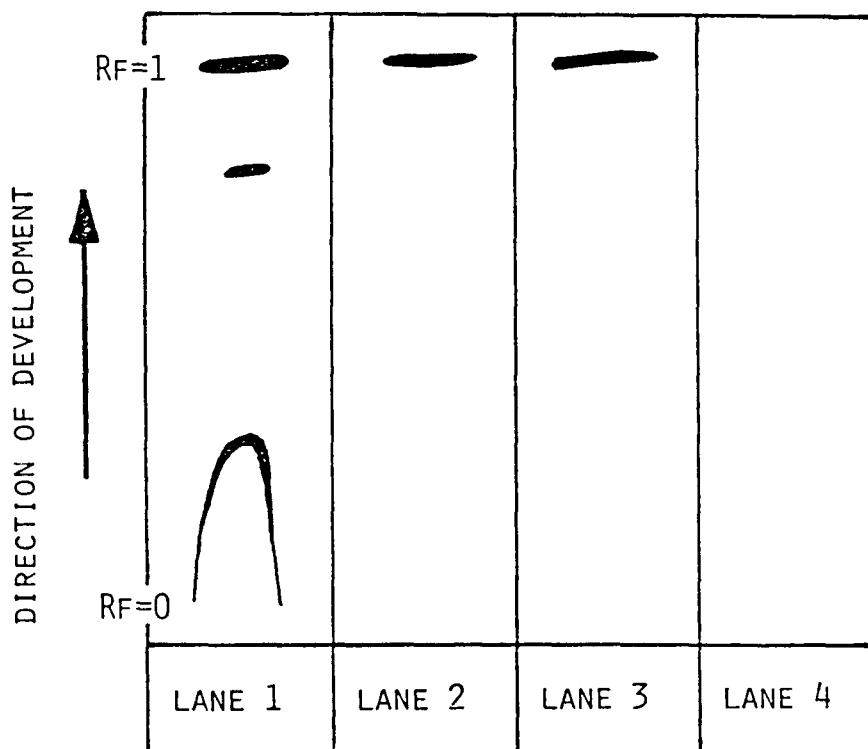


FIGURE 2    AROCLOR AND OTHER HALOGENATED HYDROCARBONS VISUALIZED ON HPTLC. A diagram of dichloromethane (100%) development of a chromatographic plate (EM Science Silica Gel 60 F254) spotted with terrestrial soil extract (lane 1); AROCLOR 1254, 1mg/ml (lane 2); aquatic sediment extract from Sheboygan River, WI (lane 3); and, reagent blank (lane 4). The plate was treated to visualize halogenated hydrocarbons, which appear as dark areas in this diagram.

EAGLE HARBOR SEDIMENT EXTRACTS
AND TA-98



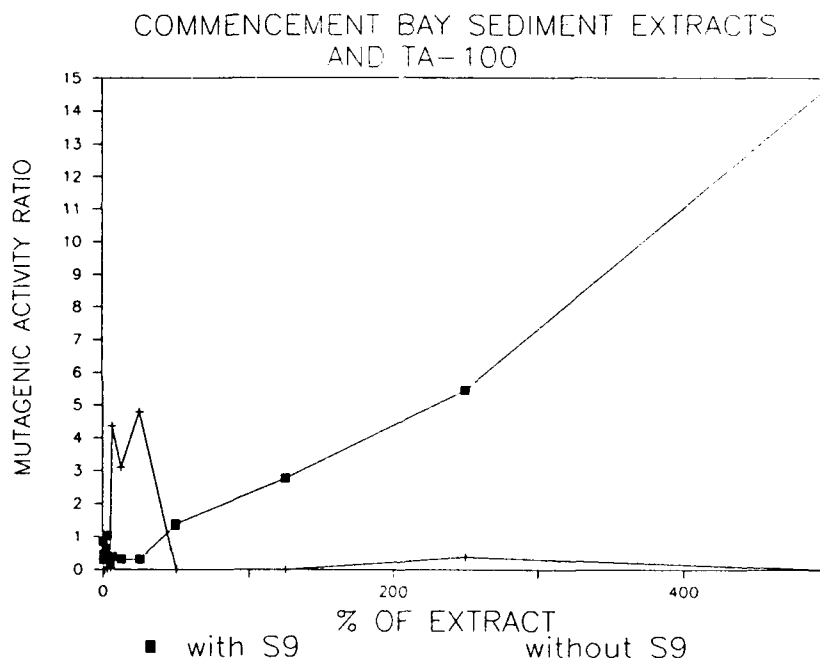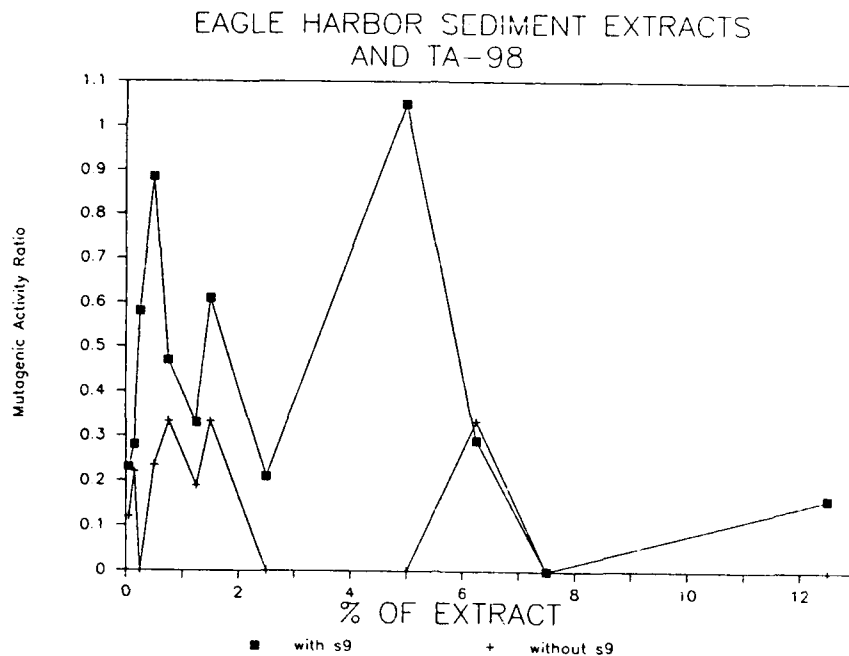COMMENCEMENT BAY SEDIMENT EXTRACTS
AND TA-100

FIGURE 3   THE EFFECTS OF SEDIMENT EXTRACT DOSE ON MUTATION RATE OF AMES  TESTER
STRAINS.  The mutagenic activity ratio = the number of mutations above background
divided by the spontaneous mutation rate (y-axis).  The extract was either diluted or
concentrated to get variations in dose expressed as % of the original concentration of
extract obtained from PSEP.  Tester strains were exposed to sediment extracts both in the
presence and in the absence of  microsomal activation S-9.  S-9 simulates the metabolic
activity found in a vertebrate liver.  In both cases extracts treated with S-9 had a dose-
dependent mutagenic response.  (a) Ames Tester strain no. TA-98 was exposed to doses
from 0 to 13% of the extract obtained from Eagle Harbor, WA.  A linear dose response is
observable below 1% concentration.  (b) Ames Tester strain TA-100 was ex-posed to doses
form 0 to 500% of the extract obtained from Com-mencement Bay, WA.  A linear dose
response is observable from 125 to 500%.

# "UV Fingerprint"
## UNKNOWN MUTAGEN
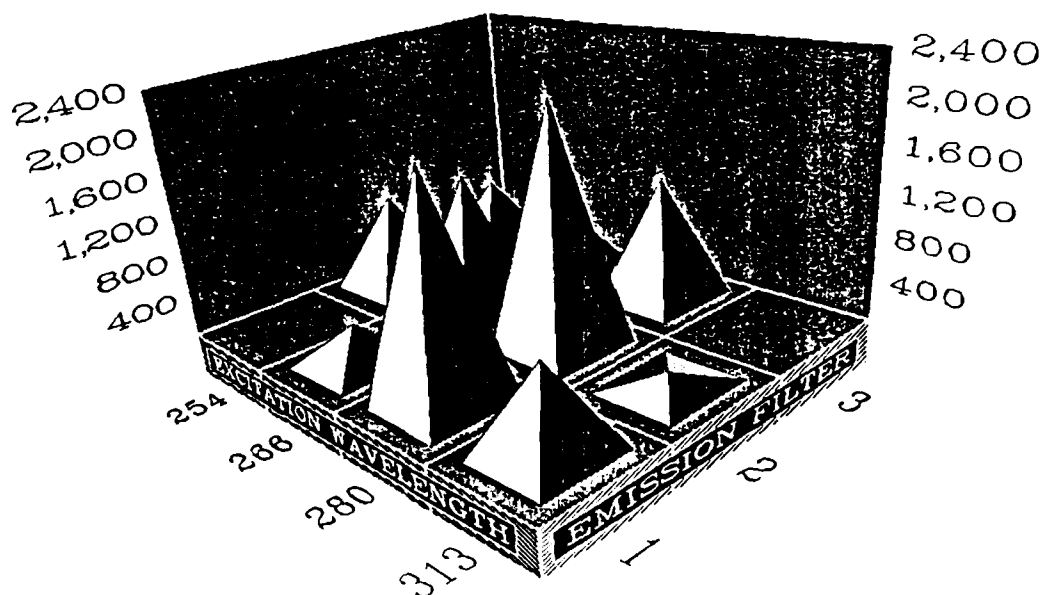


# "UV Fingerprint"
## FLUORANTHENE
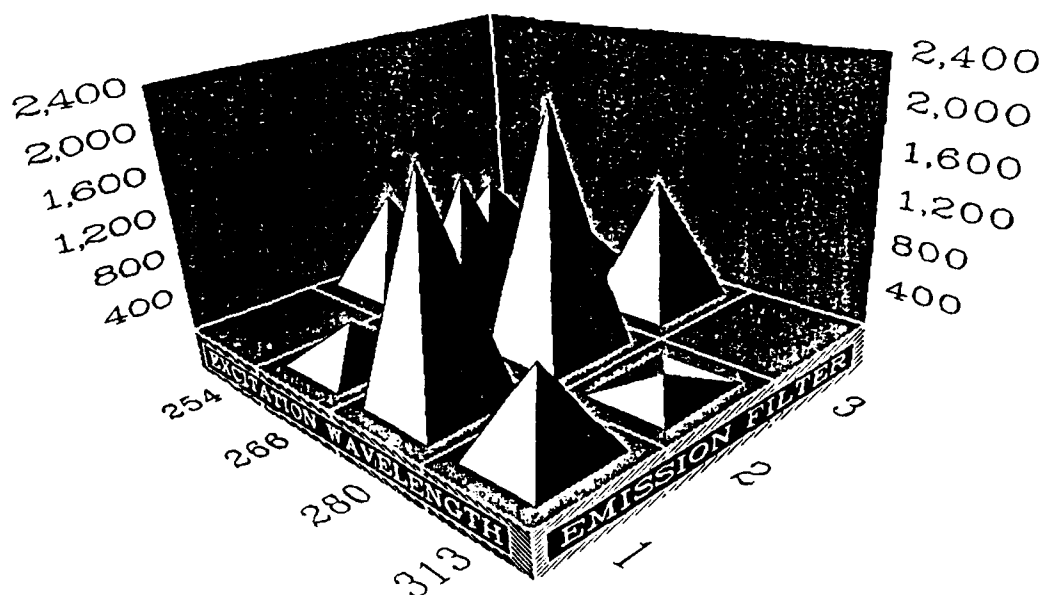


FIGURE 4    THREE-DIMENSIONAL UV FINGERPRINT OF THE UNKNOWN MUTAGEN (a) AND
FLUORANTHENE (b).  The three-dimensional fingerprint was obtained by exposing the
chromatographic spot to four wavelengths of UV light (y-axis) and measuring the relative
fluorescent reflectance (z-axis) through three different cut-off filters (x-axis) with a  scanning
densitometer.  The UV fingerprint is characteristic for each PAH and is used here to identify
the unknown mutagen as fluoranthene.

Coupling the Ames/HPTLC with further determination of chemical classes will help reduce the cost of sample analysis by substantially narrowing the field of potential compounds. Coupling the techniques has the added benefit of requiring less sample volume for initial processing, and by directing the efforts of subsequently more complete analyses, the sample can be subjected to fewer, more appropriate analyses. The number of samples submitted to any sub-sequent analysis can likewise be reduced by eliminating uncontaminated samples from further consideration.

At present, the techniques described are semi-quantitative in that a range of possible concentrations can be determined, from above the minimum concentration required for visualization, but below the concentration that causes Ames tester-strain death. Future work in this laboratory will be directed towards quantitative determinations, coupling Ames/HPTLC chemical class analysis, and increasing the number of chemical classes able to be visualized on HPTLC plates.

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CHESAPEAKE BAY THREE-DIMENSIONAL MODEL STUDY

by

Carl F. Cerco[1]

## INTRODUCTION

Chesapeake Bay is the nation's largest, most productive estuary. Pressures of population and industrialization have caused a measureable decline in aesthetic and economic resources of the Bay. Among the identified problems (USEPA,1983a,b) are eutrophication (characterized by excessive nutrient concentrations and anoxic bottom waters), decline in submerged aquatic vegetation, decline in harvest of finfish and shellfish, and toxic substance pollution.

Numerous water quality studies of the Bay have been conducted in efforts to understand causes of the decline and to formulate plans for alleviating the problems. Most recently, a three-dimensional eutrophication model was calibrated to steady-state conditions in the Bay (HydroQual,1987). The model study identified interactions between bottom sediments and overlying water as key components of the eutrophication process. Calibration of the model to observed conditions in the water column was impossible without the imposition of sediment-water fluxes as boundary conditions. The study also indicated the limitations of the steady-state approach. Extensive "tuning" of dispersion coefficients was required to match observed salinities. No estimate was attained of the time required for the Bay to respond to improvement measures.

## PLAN OF PRESENT STUDY

Previous model studies indicate several areas in which improvement is necessary in order to obtain a predictive tool for managing Bay water quality. A time-variable, three-dimensional model of transport and dispersion is required. The ability of the model to resolve vertical density stratification, which leads to bottom water anoxia, is especially important. Processes that determine nutrient recycling and oxygen consumption in the bottom sediments must be modeled explicitly. Model simulations of a time period sufficient to resolve long-term changes in Bay water quality are to be conducted.

A model package sufficient to meet these needs is presently being developed (Dortch et al.,1988). The package consists of three interacting models: a time-variable, three-dimensional hydrodynamic model; a time-variable, three-dimensional water quality model; and a predictive model of sediment nutrient and oxygen flux. The model package is undergoing calibration through a simulation of tides and currents,

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dissolved substances, and benthic fluxes observed in the complete year 1985. The models will be verified against similar data bases collected in 1984 and 1986. Once calibrated, the models will be used in a predictive mode to simulate a period sufficiently long to attain improvements in Bay water quality. This period is estimated to be five to thirty years.

## THE HYDRODYNAMIC MODEL

The hydrodynamic model (HDM) is an improved version of the model denoted CH3D (Sheng,1986). The model operates on an intratidal (less than a tidal cycle) time scale, employs boundary-fitted coordinates in the longitudinal-lateral plane, and incorporates a higher-order turbulence closure scheme to model vertical eddy transport. The intratidal time scale allows accurate prediction of currents, diffusion, and transport without the need to "tune" dispersion coefficients. Curvilinear coordinates enhance model resolution in the highly irregular geometry of Chesapeake Bay. The turbulence closure scheme ensures accurate representation of the physical processes that lead to vertical density stratification.

The model has been calibrated through simulation of Chesapeake Bay tides, currents, and salinity during a thirty-day highly-dynamic period in September 1983. Verification has been conducted through simulation of the Bay for the complete year 1985 (Johnson et al.,1989). The 1985 simulation is the largest, most lengthy application ever accomplished of a three-dimensional hydrodynamic model to an estuary.

## LINKAGE TO THE WATER QUALITY MODEL

The hydrodynamic and water quality models are operated as separate modules. Output from the HDM is written to an intermediate file that is used as input by the water quality model (WQM). This process is computationally efficient. Numerous WQM runs can be executed without recomputing the hydrodynamics. Indirect linkage of the two models presents several challenges, however. Care is required that water surface levels, flows, and diffusion processes are correctly transferred between the two models. Limits on computation time force operation of the WQM on a longer time scale than the HDM. Rigorous tests have shown that transport in the WQM, computed approximately hourly, agrees with transport in the HDM, computed at much shorter intervals.

A major technological accomplishment has been the application of the principles of Stokes' drift (Longuet-Higgins,1969) to the averaging of hydrodynamic output. The method developed allows averaging of hydrodynamic output over two tidal cycles. The WQM, employing this averaged output, maintains the transport properties of the HDM as demonstrated by a comparison of salinity computed in both models for the year 1985 (Dortch et al.,1989)

## THE WATER QUALITY MODEL

The WQM was developed especially for this project. The model schematizes the Bay into a three-dimensional matrix of interconnected,

sequentially numbered boxes. The concept is similar to the WASP model (Ambrose et al.,1986) but with several improvements. Notably, transport equations in the longitudinal and lateral directions are solved using a three-point numerical scheme, QUICKEST (Leonard,1979), that minimizes numerical dispersion. The vertical transport equation is solved using an implicit numerical scheme that allows employment of longer integration time steps than an explicit scheme.

State variables (Table 1) and processes (Table 2) were incorporated in the model based on recommendations of a workshop, attended by Bay scientists and engineers, convened for that purpose (HydroQual,1988a). The model differs from many conventional water quality models in that algal biomass is represented as carbon rather than chlorophyll a. Oxygen consumption in the water column is represented by oxidation of organic carbon rather than biochemical oxygen demand. These formulations facilitate comparisons of predictions with observations and optimize interactions of the WQM and sediment model (SDM).

Several other features of the model were also necessitated by interactions with the SDM. Algae are sorted into groups differentiated largely by the rates at which they settle to the bottom. Particulate organic matter is separated into labile and refractory fractions so that the time scale of decay in the sediments is correctly represented. Chemical oxygen demand, released by the sediments, is oxidized in the water column.

## THE SEDIMENT MODEL

The WQM and SDM are run interactively rather than linked indirectly as the HM and WQM. The sediments are schematized as two layers, an aerobic layer in contact with the water column, and a deeper anaerobic layer. SDM segments directly underly WQM segments and the schematization may be viewed as an extension of the box model concept from the water column into the sediments.

The SDM represents three fundamental processes: net settling of particles to the sediment; diagenesis (decay) of organic matter in the sediment; and flux of substances between sediments and water column. SDM kinetics are a development of concepts expressed by DiToro (1986). Fluxes predicted by the model and the processes that induce the fluxes (Table 3) are specified based on recommendations of a workshop convened for that purpose (HydroQual,1988b).

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## ACKNOWLEDGEMENTS

## TABLE 1. WATER QUALITY MODEL STATE VARIABLES

Temperature

Salinity

Iron and Manganese

Diatoms

Cyanobacteria (Blue-green Algae)

Other Phytoplankton

Dissolved Organic Carbon

Labile Particulate Organic Carbon

Refractory Particulate Organic Carbon

Ammonium

Nitrate+Nitrite

Dissolved Organic Nitrogen

Labile Particulate Organic Nitrogen

Refractory Particulate Organic Nitrogen

Dissolved Inorganic Phosphorus

Dissolved Organic Phosphorus

Labile Particulate Organic Phosphorus

Refractory Particulate Organic Phosphorus

Dissolved Oxygen

Chemical Oxygen Demand

Dissolved Silica

Particulate Biogenic Silica

## TABLE 2. WATER QUALITY MODEL PROCESSES

Algal primary production, respiration, and settling

Dissolution and settling of particulate organic matter

Mineralization of dissolved organic matter

Nitrification

Exchange of phosphate and silica with iron and manganese

Exertion of chemical oxygen demand

Oxidation of dissolved organic carbon

Reaeration

## TABLE 3. SEDIMENT MODEL FLUXES AND PROCESSES

Fluxes

| | |
|---|---|
| Dissolved Oxygen | Silica |
| Ammonium | Methane |
| Nitrate | Sulfide |
| Dissolved Inorganic Phosphorus | |

Processes

Diagenesis of organic matter

Nitrification and denitrification

Sulfate reduction and sulfide oxidation

Methane production and oxidation

Partitioning of particulate and dissolved phosphorus

Partitioning of particulate and dissolved silica

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**POSTER SESSION**

# CONTINUOUS MONITORING OF STRIPED BASS EGGS
# AND LARVAE IN THE SACRAMENTO RIVER,
# CALIFORNIA: A POTENTIAL MANAGEMENT TOOL

by

James F. Arthur[1], Melvin D. Ball[1], Lloyd Hess[1],
Charles R. Liston[2] and James F. LaBounty[2]

During spring of 1989, the Bureau of Reclamation began developmental research on methods to continuously monitor densities of striped bass eggs and larvae at a in point on the Sacramento River below major spawning grounds. Recent declines in striped bass numbers in the San Francisco Bay-Delta Estuary have been attributed in part to losses of eggs and larvae through water diversions that intercept the early life stages before they enter favored nursery zones downstream. More precise knowledge of the timing of occurrences and peak densities of eggs and larvae moving downstream will help water managers employ protective measures such as closure of water diversion gates or pumps at critical periods.

Three sampling methods were tested for their sampling adequacy, personnel requirements and ability to operate unattended for at least 12 hours. Two devices were run off the intake from a low-head, high volume submersible pump and included a cylindrical upflow tank, and stilling box. the third sampler was a modified plankton net capable of sieving water unattended for 24 hours. All water sampled was filtered through $500\mu$-mesh screens or nets. Sampling began on April 18, 1989, and ran continuously 24 hours a day, 7 days a week through mid-June.

All three samplers effectively documented peaks and occurrences of egg and larval densities. Approximately 70 percent of striped bass spawning occurred in two distinct 1-week peaks, 1 month apart. Spawning was initially triggered by increasing water temperatures. River flow was the major factor determining how rapidly the eggs were dispersed from spawning sites and the level of egg and larval development at the collection site. Estimates of larval abundances effectively sampled by the plankton net system.

Further research is planed for 1990 and will include two additional sites and an actual operational study on the Sacramento River to test effects on timed-water diversions on egg and larval transport into downstream nursery zones.

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# TECHNIQUES FOR EMERGENCY WATER PLANNING

by

Joe Wanielista, P.E.[1]

Maintaining an adequate supply of quality water during emergency conditions is an important component of municipal, regional, or statewide emergency preparedness planning which is often overlooked. During an emergency, water may be required for a number of purposes as for public consumption. On this poster, results will be presented from two studies of emergency water planning conducted by the Detroit District of the U.S. Army Corps of Engineers. More specifically, several techniques developed to facilitate emergency planning for water systems at both the local and statewide levels and presented. These techniques include: a general hazard analysis procedure; EMERGE, a micro-computer model for assessing water supply source reliability; a critical component analysis procedure for assessing system vulnerability to major component failures; a water supply system database and database management system developed to assist in emergency planning and response; and a predictive to assist in estimating the operating requirements of water supply systems under both normal and emergency conditions

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# RECOVERY

## by

## Carlos Ruiz[1]


RECOVERY is a PC-based screening model of long-term fate of in-place contaminated sediments in aquatic environments. It predicts the concentration of a contaminant in the water, the mixed sediment layer, and in the deep sediments over time. The flux of the contaminant from the sediments into the water and from the water to the atmosphere is also predicted.

The total number of years for which the model is run is determined by approximating the time required for the toxic concentration in the water to decrease to 10% of the maximum value achieved, up to a maximum of 100 years.

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# CE-QUAL-W2

## by

## Thomas M. Cole & Toni Schneider[1]

CE-QUAL-W2 is a longitudinal and vertical hydrodynamic/water quality model that can be applied to rivers, lakes, reservoirs, and estuaries. the model has been extensively rewritten for use on 80386-based PC's. The original test application of the model to DeGray Lake took approximately 25 CPU hours on a VAX 11/750 to simulate 251 days. The latest version of the model took 1.5 CPU hours on a 25 Mhz 80386 PC equipped with a Weitek math coprocessor. The input and output of the model have been extensively reformatted in order to make the model easier to use. In addition, the model no longer requires the user to rewrite portions of the code for a particular application. Future improvements to the model will include a windowing interface with online help for pre- and postprocessing.

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# A MULTI-LABORATORY QUALITY ASSURANCE PLAN

by

John C. Ellis[1]

The Bureau of Reclamation's role in chemical testing is changing with increased participation in NPDES, SDWA, RCRA, CERCLA, and SARA programs. Seven Reclamation laboratories provide analytical services related to these programs and others. The objectives and elements of Reclamation's quality control/quality assurance (QA/QC) plan are presented in a poster session. The Quality Assurance Plan will establish Reclamation guidelines for statements of work, contracted analyses, sampling and sample handling, instrument recordkeeping, and laboratory performance audits. Interlaboratory activities are co-ordinated through the Bureau of Reclamation and regional Quality Assurance officers.

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# WATER RESOURCE ENGINEERING AND ACID MINE DRAINAGE
## IN THE UPPER OHIO RIVER BASIN

by

Michael Koryak[1]

Acid mine drainage from bituminous coal mines has been the greatest single water pollution problem in the upper Ohio River Basin. Thousands of miles of streams within western Pennsylvania, northern West Virginia, western Maryland, and southeastern Ohio have been degraded by an acid mine drainage (AMD) load that, until recent decades, was equivalent to more than a million tons/year of sulfuric acid. Severe AMD pollution caused damage by corroding pipes, pumps, boats, gates, and navigational aids. The acid and associated mineralization, along with frequent gross heavy metal pollution, degraded the aesthetic and recreational value of local waters. AMD suppressed and often totally eliminated aquatic life in impoundments and along substantial reaches of major rivers. AMD also caused numerous and serious domestic and industrial water supply problems.

Because of the extent and magnitude of the problem, AMD considerations have had a significant influence on many aspects of water resource development engineering in the upper Ohio River Drainage Basin. In some areas, AMD necessitated the use of special construction techniques and corrosion-resistant materials, and it increased and complicated maintenance problems.

AMD has also exerted a major influence in the planning, design, and operation of large civil works engineering projects. Operation schedules to moderate low-flow AMD degradation extremes were developed as integral parts of Corps of Engineers reservoir projects. While dramatic progress has recently been made in AMD abatement, these projects and operations continue to provide very substantial AMD mitigation benefits.

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# WATER QUALITY INVESTIGATIONS
## FOR THE CHICAGOLAND UNDERFLOW PLAN RESERVOIRS

by

Environmental Engineering Section
U.S. Army Engineer District, Chicago

The Chicagoland Underflow Plan (CUP) is the Federal interest portion of the Tunnel and Reservoir Plan (TARP) developed by the Metropolitan Water Reclamation District of Greater Chicago (MWRDGC). TARP was designed to alleviate urban flooding and water pollution problems resulting from the overwhelming of the combined sewer system of Chicago and the surrounding area. TARP was divided into two phases. Phase I is primarily concerned with water quality and consists of 110 miles of tunnels, near surface collectors, drop shaft systems, plus appurtenant works. Construction of TARP Phase I is nearly completed. TARP Phase II is primarily concerned with flood control and consists of 22 miles of tunnels, near surface collectors and drop shaft systems, and two on-line and three terminal reservoirs, plus appurtenant works. The terminal reservoirs of TARP Phase II are being designed and constructed by the Corps.

The three reservoirs vary in size from 1,050 acre-feet for the O'Hare Reservoir to 32,000 acre-feet for the McCook Reservoir. The reservoirs will receive and store a combination of sanitary sewage and stormwater runoff that would have been discharged to area watercourses through combined sewer overflows. Since sanitary sewage would be present in the influent, problems were anticipated with the development of unacceptable odors associated with the anaerobic breakdown of organic matter in the reservoir. To alleviate these potential odor problems an aeration system was planned for the reservoir.

The proper design for the aerator system depended on an analysis of the characteristics of the influent, specifically the amount of oxygen necessary to keep the reservoir water aerobic. A literature search concluded that similar reservoirs of this size did not exist. A zero-dimensional computer model was developed by the WES through a WOTS request and was used to simulate the fate of the dissolved oxygen resources in the O'Hare Reservoir. Construction of the O'Hare Reservoir is scheduled to begin in FY 90.

The McCook Reservoir is a much larger and more complex problem than the O'Hare Reservoir, and the water quality modeling effort is being expanded for the design of the aeration system. Dr. Richard Price (CEWES-HS-R) has supervised the modification of the 1-D computer model CE-QUAL for use at McCook. Results from this model study, due in February 1990, will be used to determine the need for development of a full, 3-D hydrodynamic model of the reservoir.

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**16. SUPPLEMENTARY NOTATION**
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| 17. COSATI CODES | | | 18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number) |
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**19. ABSTRACT (Continue on reverse if necessary and identify by block number)**

⊅ This ⊃ pesented at the seminar includes,
A COLLECTION OF 34 PAPERS INCLUDING TOPICS ON RESERVOIR AND RIVERINE CASE STUDIES, AND COASTAL AND ESTUARINE CASE STUDIES. ALSO INCLUDED ARE 2 PAPERS FROM THE PLENARY SESSION AND 7 ABSTRACTS REGARDING POSTER SESSION DISPLAYS. Keywords: Water quality/ resources; Inland waterways; Coastal regions/water; Lakes/rivers; Dams/reservoirs/drought; Sea level; Estuaries/suspended sediments; Monitoring/fishes; Water conservation; Dredging/contaminants/ disposal; Damage assessment; Water pollution/ analysis; Data bases; Dioxins/toxicity; (over)

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